THE EVALUATION OF SHORE PROTECTION STRUCTURES USED FOR EROSION CONTROL AT LAKE PUKAKI, NEW ZEALAND

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by

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Frontispiece

Abstract

This thesis investigates the shore processes of Lake Pukaki to assess the suitability and performance of existing and potential future options for management of shore hazards. Following two successive raisings of the water level in the past sixty years, Lake Pukaki's shoreline has undergone extensive erosion. Since mid 1987 until early 1988, shoreline erosion control structures (i.e. rock revetments, gabion baskets and groynes) were constructed and maintained to protect sections of road and other assets from further encroachment of the lake shoreline.

The use of the RBR XR-620 pressure sensor in this study marks the first occasion when wave statistics were measured via instrumentation at Lake Pukaki. The mean significant wave height (H_s) identified for this study was 0.53 m, while the maximum recorded wave height was 1.84 m. Similar to other alpine lakes, Lake Pukaki has characteristics of steep plunging waves. The LAKEWAVE wave hindcasting model is used to describe the wave environment about the Lake Pukaki shoreline in terms of its optimum energy potential. The maximum wave height and wave period values estimated by LAKEWAVE have been found to compare well with measured wave statistics

Under the current wave climate, experienced during this study period (July 2010 – February 2011), the majority of the assessed rock revetments seem to be performing well. The Hudson and Van der Meer formula seemed to predict respectable stability thresholds that agree with what was observed in the field. The revetment at Site 3 is the biggest concern in terms of performance based on field observations.

The short-period high-magnitude storm events, eventuating from a strong north/northwest wind flow, that coincide with high lake levels tend to cause the most significant erosion along the shoreline at Lake Pukaki and have a major influence of riprap stability. Other environmental factors including the steep nearshore profile, the glacial till backshore, groundwater and precipitation were identified as controlling factors leading to the success or failure of the shore protection structures.

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There are many people that have helped me to complete this thesis and it is with great appreciation that I thank you all for your assistance.

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Many thanks to Neil Sutherland at Meridian Energy, for providing me with the lake level data and giving me access to Meridian's library catalogue. I would like to thank Opus International Consultants for granting me permission to use the shoreline investigation reports as part of my work. I hope the results from this research help to assist in any way towards the future management of the Lake Pukaki shoreline.

None of the fieldwork could have been accomplished without the help of my field assistants. In particular, my brother Tim Mathewson, who helped out with the beach surveying. Special thanks also go out to Ben Snowdon and Fiona Campbell. Thanks for keeping me company during the car ride and in the field, even when the weather turned bad. This may have paid off when having to watch me retrieve the submerged wave equipment while it was snowing.

I would also like to thank Ian and Nicky Hayman and Ross and Helen Ivey for permitting me to leave the weather stations on your property. Also, for allowing me to access them whenever I needed to and protecting them from nearby stock.

Financial assistance for this study was provided by the Geographical Society Research Grant. This helped me to afford numerous trips down to Lake Pukaki and the occasional lunch special at the lunch bar in Twizel, which never seemed to change.

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Δ	Relative buoyant density of rock $((\rho_r/\rho_w)-1)$
ΔT_A	Air temperature (°C)
ΔT_W	Water temperature (°C)
ΔT_{AW}	Air-water temperature differential (°C)
α	Revetment slope angle (°)
α_b	Breaker wave approach angle (radians and °)
β	Beach slope angle (°)
C	Wave runup coefficient
C_g	Wave group velocity (ms^{-1})
d	Water depth (m)
D	Wind duration (hours)
D_n	Nominal diameter (m)
D_{n50}	Median D_n of rock grading given by 50% on the mass distribution curve (m)
E	Wave energy (Ns^{-1})
F	Fetch (m)
FAS	Fully arisen sea, fully developed sea
FoS	Factor of safety
f_p	Peak frequency (Hz)
g	Acceleration due to gravity (9.81 ms^{-2})
H	Wave height (m)
H_b	Wave breaker height (m)
H_d	Design wave height (m)
H_o	Deepwater wave height (m)
H_{max}	Maximum wave height (m)
H_{rms}	Root mean square wave height (m)
H_s	Significant wave height (m)
H_{10}	Average height of the highest one-tenth waves (m)
H_{50}	Average wave height of the fifty highest waves reaching a structure in its life (m)
k	Coefficient (LEXSED formula)
K_D	Stability coefficient (Hudson equation)

K_R	Refraction coefficient
K_S	Shoaling coefficient (Hudson equation)
L	Wave length (m)
L_o	Deepwater wave length (m)
M	Mass of armour unit (kg or tonnes)
M_{50}	Median mass of rock grading given by 50% on the mass distribution curve
N	Number of waves
P	Permeability coefficient; wave power (Nms^{-1} m^{-1})
P_A	Atmospheric pressure (dbar)
P_R	Recorded pressure reading from RBR XR-620 (dBar)
P_W	Water pressure (dbar)
Q	Longshore sediment transport rate (m^3 s^{-1} and m^3 h^{-1})
R	Correlation coefficient; wave run-up elevation (m)
R^2	Coefficient of determination
$R_{2\%}$	2% exceedence of all runups (m)
R_T	Amplification factor
S	Damage level
T	Wave period (s)
T_c	Wave crest period (s)
T_s	Significant wave period (s)
T_z	Zero crossing period, mean period of all the waves in the record (s)
U	Wind speed (ms^{-1})
U_A	Adjusted wind speed (ms^{-1})
U_o	Observed wind speed (ms^{-1})
U_{10}	Wind speed at adjusted 10 m reference height (ms^{-1})
V_{SW}	Swash velocity (ms^{-1})
Z	Height of the climate station above the water level (m)
ρ_r	Density of rock (kg/m^3)
ρ_w	Water density (kg/m^3)
ε	Spectral bandwidth
ξ_o	Deepwater iribarren parameter, surf similarity parameter
$ø$	Angle between the wind and off wind direction (°)

Chapter 1

INTRODUCTION

1.1 Thesis Statement

This thesis investigates the shore processes of Lake Pukaki in order to assess the appropriateness and performance of existing and potential future options for management of shore hazards. Following two successive raisings of the water level in the past sixty years, Lake Pukaki's shoreline has undergone extensive erosion. Erosion is not the current management problem; it is the impact on assets in use by the public, located behind the shoreline. This problem has lead to the implementation of shoreline management strategies, involving the construction of shore protection structures. To appreciate the energy potential for causing widespread erosion of Lake Pukaki's shoreline, an understanding of the nearshore processes at work is essential.

Lake Pukaki is the largest hydro-electric storage lake in New Zealand, with a current storage capacity of 1679.65 GWh. Along with Lake Tekapo (782.36 GWh) it provides over half of New Zealand's hydroelectricity storage capacity (Meridian, 2010). It is an upper member of the Waitaki Power Scheme under the current management of Meridian Energy Limited. The lake level prior 1946 was 485 msl (metres above mean sea level) and its outflow was at its southern end, into the Pukaki River. To maximise power generation the storage capacity of the lake has been increased by damming across the southern shore. In 1951 the Pukaki low dam was constructed and raised the water level by 6 m to 491 msl (Kirk, 1988). During 1977 - 1979 the water level rose a further 33.4 m, following the construction of the Pukaki high dam in 1976, with a crest height of 61 m. The current lake has an operating range of 14.45 m, the level which can be artificially raised or lowered, with a maximum level of 532.65 msl and a minimum level of 518.2 msl (Single and Bunting, 2003). Following the second elevation of the lake level in the 1970s, at the higher end of its range, the shoreline length increased by more than 30%, the fetch length of the lake, that is the area over which wind can generate waves, increased by 50%, and the water surface area rose by about 64% (Kirk, 1988).

The artificially raised water level and a fluctuating water surface initiated a new evolution sequence of shoreline development in Lake Pukaki, which involves erosion of the nearshore profile into a new nearshore platform. In concurrence with the intense wave activity, beach steepness and low erosive resistance of shoreline materials, this has caused widespread erosion of the surrounding shore, and a net landward retreat of the shoreline. Comparing his results to those of Kirk (1988), Allan (1991) concluded that the rate of erosion appeared to have declined since 1988. The erosion rates were high following the initial water level raising as the shoreline adjusted to a new equilibrium profile.

As the shoreline continues to retreat assets behind the shoreline have become under threat. The road network surrounding the shoreline of Lake Pukaki is at risk from erosion along various lengths and locations. State Highway 8 flanks the southern shore, and serves as the major highway in the Mackenzie Basin. State Highway 80 stretches along the western shore and continues for 55 km before reaching Mount Cook Village. Braemar and Hayman Road extend along the eastern shorelines margin. These two metal roads act as access roads to Mount Cook Station, Tekapo B Powerstation and the Canal Road to Tekapo.

Since mid 1987 and early 1988 shoreline erosion control structures (i.e. rock revetments, gabion baskets and groynes) have been constructed and maintained in an effort to protect sections of road and other assets from further encroachment of the lake shoreline. These works have varied in the degree of success as shore protection structures, with many requiring repair, and some failing completely (Opus, 2009). Ongoing erosion in the vicinity of existing structures presents a practical problem for the managers of the lakeshore environment.

The literature affiliated with lacustrine research is analysed in further detail in the following section. The Lake Pukaki field site is discussed in terms of its setting, morphology, geology and climate. The regional geology of the Mackenzie Basin is examined and the sediments surrounding the shoreline of Lake Pukaki are briefly investigated with respect to their origin and characteristics. The overall approach for this thesis, including the aims and structure, is outlined in the last section of this chapter.

1.2 Lake Shoreline Research

1.2.1 Introduction

The majority of literature addressing coastal processes and management is associated with open coast settings. In contrast, the literature affiliated with lakeshore environments is very limited. Dawe (2006) stated that, there are clear differences between coastal and lakeshore environments.

In open coast settings the main limiting factors controlling wave growth tend to be the wind velocity and duration. For an enclosed water body, such as a small lake or estuary, wave development is also commonly restricted by the fetch length. Fetch length is the area over which wind generated waves can form. Accordingly, restricted fetch environments can only generate small, short period, low energy waves that are characteristically steeper and more erosive in nature (Pickrill, 1985). Due to their constrictive size, lakes are also unable to develop swell waves as there is not enough space for wave dispersion to occur across.

The nearshore process environment for a restricted water body is simplified in comparison to an open coast setting. Waves tend to only approach the shoreline one direction at a time, which is inherently controlled by the wind (Dawe, 2006). In alpine settings the surrounding topography can exert a powerful constraining influence on the wind direction by channelling flow down the valley axes. In particular, this is significant for narrow alpine lakes where the shoreline is only subject to waves from one or two dominant directions.

Oceanic beaches, unlike lakes, undergo one (diurnal) or two (semidiurnal) tidal fluctuations each day. The small relative size of lakes in contrast to oceans means that the combined effects of the gravitational forces exerted by the Moon, the Sun and the rotation of the Earth are incapable of causing such deformation of the water surface. However, lake water levels do fluctuate naturally on a seasonal and annual basis. The water level of a lake is controlled by the inflow of rivers, precipitation and evaporation (Dawe, 2006). For power generation purposes, lakes can be dammed and the water level (operating range) can be artificially controlled. For example, throughout autumn the water level can be raised to combat the high power consumption during the winter months. Fluctuation of the water level allows waves to act on different parts of the beach profile. In contrast to sea levels affected by tidal cycles on an oceanic beach, lake levels can be situated at a certain elevation for days, weeks or months. Incident wave energy can therefore be concentrated on one section of the beach profile for a prolonged time (James *et al.* 2002). For this reason, Dawe (2006) states that lake beaches have greater difficulty developing an equilibrium profile compared to oceanic beaches.

To gain a comprehensive understanding of the current coastal management problem around the Lake Pukaki shore, the literature associated with the lakeshore environment needs to be assessed. Findings from research and technical reports carried out at Lake Pukaki are pertinent for this thesis. Research fulfilled on other New Zealand alpine lakes can provide background and examples for a similar environment. Other lacustrine research which has been accomplished overseas also provides to be useful. However, differences and similarities need to be outlined and justified when making comparisons to the Lake Pukaki setting. The literature surrounding lakeshore processes and management is discussed further in the following section.

1.2.2 Alpine Lake Research in New Zealand

Alpine lakes are classified as lakes or reservoirs residing at high altitudes that have originated from glacial processes. There are two hundred and sixty eight glacial lakes in New Zealand, nine of which are in excess of 50 km^2, residing within the Southern Alps of the South Island (Irwin, 1975). The majority, of the nine, are utilised for hydro-power generation, with a few having been modified by dams or other control structures (Allan, 1991).

According to Allan (1998), and revisited by Dawe (2006), the literature on lacustrine research can be divided into three broad sections. The first area concerns the morphodynamics of lake shorelines with emphasis on evolution and stability. The second area pertains to lakeshore management. The third and final area concerns the limnology of lakeshore environments including studies of sedimentation (Irwin, 1971; 1978), water turbidity (Rowe *et al*. 2003) and internal temperature structures (Zawar-Reza *et al*. 2004). The first and second section are discussed in further detail below, however the literature surrounding the limnology of lakeshores will not be because it is not of major concern regarding this thesis.

Due to a lack of publication there is a limited amount of lakeshore literature available within New Zealand. Very little work on lakeshore processes has been carried out in the past five years, since Dawe's (2006) Ph.D. thesis at Lake Coleridge, examining the processes of longshore sediment transport in the swash zone of a mixed sand and gravel shoreline. Prior Dawe's (2006) work was Allan's (1998) Ph.D. thesis at Lake Dunstan, investigating the morphological development of a new lacustrine shoreline created behind a hydro-electric dam. Both Allan (1998) and Dawe (2006) determined that lake waves were typically small (0.07 – 0.57 m), prevalent by high frequency waves (1.43 – 2.9 s), were characteristically steep (0.01 – 0.07), making them extremely erosional at the shore. Allan (1998) expresses there is a need for detailed process work to be carried out on lakes, during high energy storm events because this is when beach change is most pronounced.

Research completed at Lake Pukaki by Bunting (1977), a multifaceted study carried out by Kirk (1988) on shoreline change and management, and a Master's thesis by Allan (1991) provide an additional basis for examining the physical processes at work on the lakeshore. Bunting's (1977) engineering geological investigation considers the shoreline stability of Lake Pukaki. According to Bunting (1977), the southern shore at Lake Pukaki annually receives the largest amount of wave energy (1.7×10^{-9} kg.m/m of shore) while the western and eastern shores annually receive one half (on average) and one sixth respectively of that amount of wave energy. The annual amount of wave energy reaching the southern shore has likely increased because Bunting's (1977) investigation was completed prior the raising of the water level during 1977-1979. Kirk (1988) suggested a minimum threshold below which waves were regarded as unimportant for lakeshore erosion. This lower limit was held to be all winds below Force 4 (5.7 – 8.8 ms^{-1}). There has been a reliance on wind data, utilised for wave predictions, from weather stations that are not located along or near the lakeshore (Kirk, 1988; Kirk *et al.* 1996, 2000). For this research, weather stations are installed in close proximity to Lake Pukaki in order to give an exemplary understanding of the wind regimes role in wave generation at the lake. Following on with work carried out by Kirk (1988), Allan (1991) measured wave statistics during short period, high magnitude north/northwest wind events. Allan's (1991) thesis justifies the importance of measuring air/water differences when forecasting waves as opposed to using the amplification factor accepted by Kirk (1988). He stressed that predicted and measured waves need to be correlated to justify the applicability for predicting waves at Lake Pukaki and other New Zealand lakes. However, Allan (1991) measured waves visually, which is less accurate than a wave recording device. In this thesis the author recorded wave characteristics using a RBR XR-620 submersible data logger and a WG-30 capacitance wave staff, which has been successfully used in previous lacustrine studies in New Zealand (Allan, 1998; Dawe, 2006).

Studies conducted on other South Island alpine lakes including Lake Hawea (Kirk *et al*. 1996, 2000), Lake Coleridge (Kirk and Henriques, 1986; MacBeth, 1988; Worthington, 1989) and Lakes Te Anau and Manapouri (Pickrill, 1976; Pickrill, 1985) are also significant in considering the process context. They represent a steadily developing knowledge of lakeshore processes and an increasing measure of management understanding.

The majority of lakeshore literature that has arisen in the past 5 years is in terms of lakeshore management. Although, the literature associated with lakeshore management on New Zealand's glacial lakes is limited. The Ministry of Environment publication "Lake Manager's Handbook – Lake level management" (James *et al*. 2002) presents an overview of lakeshore management issues with regard to lake level changes and shoreline erosion. A large proportion of this literature is in the form of unpublished reports, compiled for local councils and power companies, which consider specific issues for lakes on hydropower schemes. For example, Lake Hawea (Kirk *et al*. 1996; Single and Kirk, 2001), Lake Manapouri and Te Anau (Kirk, 1985; Kirk and Single, 1998), Lake Pukaki (Kirk, 1988; Bunting *et al*. 2003; Opus, 2000; Single and Kirk, 2003), Lake Coleridge (Henderson, 1994; Kirk and Allan, 1995) and Lake Monowai (Kirk, 1992; Dawe and Single, 2001).

The major findings from the literature surrounding lakeshore processes and morphology that are pertinent to this thesis will be reviewed in Chapters 2 and 3. More detailed reviews specific to lakeshore management will be incorporated in the findings of this study in Chapters 6 and 7.

1.2.3 Other Lacustrine Research

The majority of studies that investigate lacustrine environments are associated with the Great Lakes of Northern America. Less work has dealt with lacustrine nearshore processes, with a large part being focused on beach change and management. Wave conditions within the Great Lakes are strongly influenced by fetches aligned with the dominant directions of storm winds (Resio *et al.* 2002). With fetch lengths usually exceeding 100 km, waves can develop to heights of 4 to 7 m (Buddecke, 1973). The Great Lakes shore morphology is characteristic of steep bluffs developed in cohesive till deposits, fronted by narrow beaches of coarse sand and gravel (Davidson-Arnott and Keizer, 1982; Davidson-Arnott, 1986). A large number of publications have arisen within the last 50 years on these environments in terms of assessing rates of shoreline erosion (Davidson-Arnott, 1986; Buckler and Winters, 1983) and the implementation of shore protection structures (Davidson-Arnott and Keizer, 1982; Carter *et al*. 1986; Lawrence, 1994). In contrast, there is very little work dealing with lakeshore processes in the Great Lakes. Unlike in small lakes or reservoirs, the Great Lakes of North America hydrodynamically behave as small oceans (Pierce, 2004) and are large enough to experience tidal fluctuations.

Pickrill (1976) and MacBeth (1988) assumed that the Great Lakes of Northern America had resemblances that were more typical of open coasts. Both Kirk (1988) and Allan (1998) disagreed with their views and argued that comparisons can be made with the alpine lakes of southern New Zealand. It is important to acknowledge that there are renowned differences between the lacustrine geomorphology of the Great Lakes and the alpine lakes of southern New Zealand (Table 1.1). Consideration of such differences (i.e. fetch lengths, shore morphology and shore types) must be taken into account when making use of findings from the Great Lakes or from other international studies (Allan, 1998).

Table 1.1: Comparison of the lacustrine environments of the Great Lakes of Northern America and the South Island, New Zealand Alpine Lakes. Reproduced from Allan (1998, Table 2.1, p 22).

Location	Fetch Lengths	Wave Height	Lake Levels	Shore Types and Beach Material	Beach Morphology
Great Lakes of Northern America	Typically > 100 km → 600 km Lake Superior	Waves formed in response to prevailing winds from the west and southwest. → 7.6 m Lake Superior → 4.2 m Lake Michigan → 4.0 m Lakes Erie, Huron and Ontario (Buddecke, 1973)	0.3 – 1.5 m Important factor is not so much the magnitude of lake level increase but the length of time the water levels are high. e.g. during 1985 – 1986, lake levels were high for one year.	Extremely varied. 10 identified shore 'types'. Main Focus: - sandy shores - cohesive till shores	Sandy Shores: Upward concave gently sloping (0.2°) bar type 'dissipative' form. Beach profiles are typically wide. → 350 m Cohesive Shores: Convex steep sloping linear form. Upward concave steep sloping linear foreshore.
South Island, New Zealand Alpine Lakes	Typically < 50 km. → 75 km Lake Wakatipu	Waves formed in response to topographical channelling of winds along valley axes. Important winds from the north, northwest and southwest. Few measurements. → 0.5 m Lakes Manapouri and Te Anau (Pickrill, 1976) → 0.07-0.57 m, Hmax = 1.05 m Lake Dunstan (Allan, 1998) → 0.20-0.35 m, Hmax = 0.85 m Lake Coleridge (Dawe, 2006)	Highly variable and dependant on the particular role of the lake. Highest levels generally occur over summer and tend to coincide with periods when storm incidence is elevated. 13.8 m – Lake Pukaki 8.8 m – Lake Tekapo 8.0 m – Lake Hawea 5.0 m – Lake Coleridge 1.8 m – Lake Manapouri 1.2 m – Lake Te Anau	Extremely varied. 13 identified shore 'types' Main Focus: - mixed sand and gravel shores (Dawe, 2006). To a lesser extent: - sandy shores (Pickrill, 1986) - cohesive till shores (Kirk, 1988; Allan, 1991).	Despite the variability in shore types, beaches tend to exhibit a simple three element 'reflective' type morphology. Consists of: - steep narrow foreshore (7 - 8°) - less steep nearshore shelf (5 - 8°) -extremely steep offshore face (25 – 32°).

Flathead Lake, Montana, USA and Loch Lomond, Scotland, UK, are both alpine lakes and demonstrate similarities to those of the South Island. Lorang *et al*. (1993) examines shoreline erosion at Flathead Lake, resulting from a redistribution of annual wave energy due to 50 years of regulated lake levels. Lorang *et al.* (1993) recommends that by changing the timing and increasing the amount of lake level drawdown, would result in a reduction of lake wide shoreline erosion because there would be a decrease in the amount of annual wave energy acting on the upper end of the operating range. Recent studies completed by Pierce (2004) and Hansom and McGlashan (2000) investigate the major factors leading to widespread erosion at Loch Lomond and provide recommendations for shore management strategies. Hansom and McGlashan (2000) aimed to develop a management strategy that allows minimal intervention to the natural processes and so guarantees that loch shore processes, landforms and habitats can adjust towards a dynamic equilibrium. To improve our understanding of lacustrine systems and for better management Pierce (2004) believes that lake shores need to be recognised as coastal environments and not as separate entities.

Archipelago environments are comparatively similar to alpine lakes, with the only difference being the constrained influence on the fetch length and wave angle approach due to the presence of islands within the body of water. Tolvanen and Suominen (2005) examine three GIS based approaches for assessing openness and wave exposure in archipelago environments. Vilmundardóttir *et al*. (2010), in a study of the Blöndulón hydro-electric reservoir, Iceland, an archipelago environment, found that wave induced bluff erosion was most rapid during the first years after impoundment as the new shoreline was developed in response to the new water level. Vilmundardóttir *et al*. (2010) also recognised that bluff erosion was most active at sites with glacial till substrate, high cumulative wave power, and long fetches along the dominant wind direction.

1.3 Lake Pukaki: Field Site

1.3.1 Setting

Lake Pukaki is the largest of three roughly parallel alpine lakes, which are aligned north-south along the upper edge of the Mackenzie Basin (Figure 1.1). Lake Ohau is the smallest and is situated west of Lake Pukaki, whereas Lake Tekapo resides further east. The north-south trending axis of Lake Pukaki is roughly 29 km and its short axis averages 5.5 km in length, with a maximum of 8.5 km near the southern shore. It has a surface area of 168 km^2, has a catchment area of 1359 km^2 and a maximum depth of 99 m close to its south-western shore (Kerr, 2009). The total flow into the lake is a combination of generation flow from the Tekapo B power station (max 120 $m^3 s^{-1}$) and natural inflow (~130 $m^3 s^{-1}$) from numerous rivers and streams (Meridian, 2004). The most significant contributor in regards to natural inflow would be the braided Tasman River which flows into Lake Pukaki at its northern edge (Figure 1.2). The source for the Tasman River is the Tasman, Murchison, Mueller and Hooker Glaciers.

1.3.2 Regional Geology

The Mackenzie Basin is situated within a large tectonic depression, which was formed in the Miocene (5 - 25 million years ago) during the Kaikoura Orogeny (25 mya). Lake Ohau, Pukaki and Tekapo reside in glacial troughs last occupied by ice during the last glacial maximum (10,000 B.P.) (Barrell and Cox, 2003). The bathymetry of and topography surrounding Lake Pukaki is reflective of its glacial origin. According to Bunting (1977), the bathymetry can be explained as a “U-shaped” bath tub, with steeply dipping sides and a flat bottom. The Tasman delta front, at the northern end of the lake, steeply dips south in a line east-west through Boundary Stream (Figure 1.2). The lake bed then gently slopes southwards before rising steeply again at the southern shore. Research completed by Read (1976), Bunting (1977) and Speight (1963) provide a comprehensive overview of the regional and shoreline geology, aspects of which are summarised here and in Section 1.3.3.

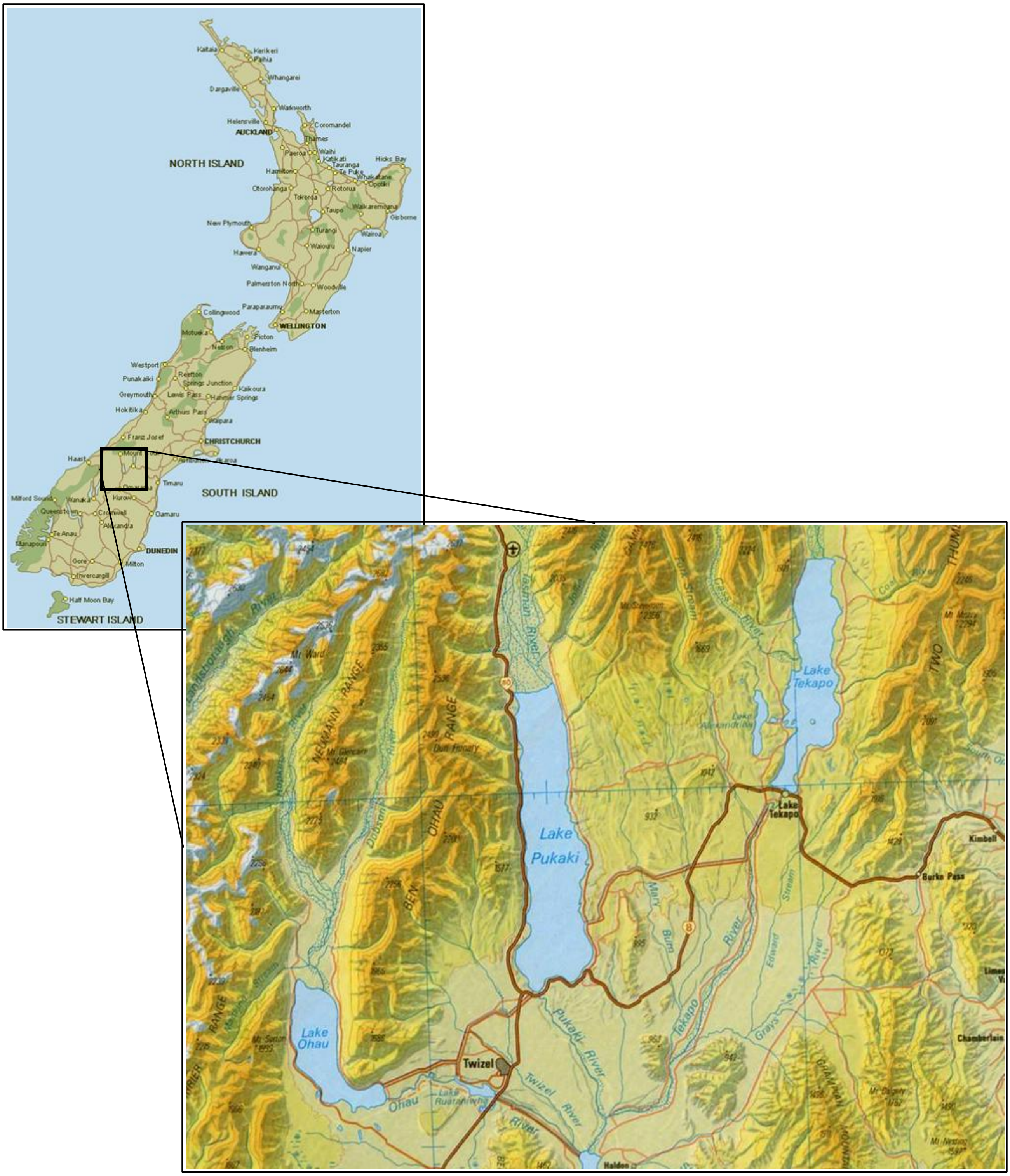


Figure 1.1: Location and extent of the Mackenzie Basin (TopoMap, 2010).

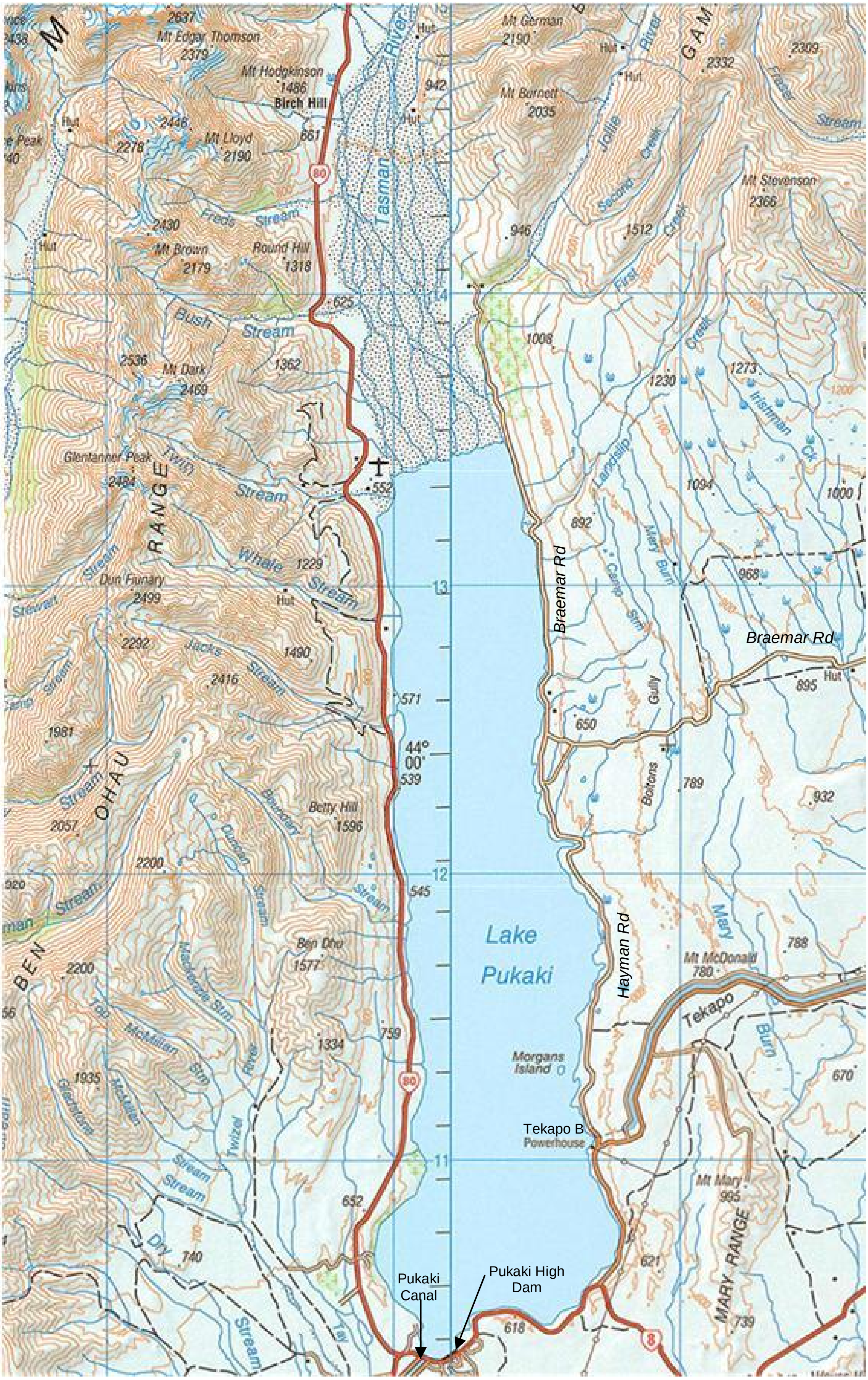


Figure 1.2: Map of Lake Pukaki (TopoMap, 2010).

The regional geology map for Lake Pukaki is reproduced from Bunting (1977) in Figure 1.3. The current water level is considerably higher than what the regional geological map by Bunting (1977) depicts, as it was developed prior the second raising of the water level between 1977 and 1979. The water level has submerged a larger proportion of the Tasman delta and now resides as far north adjacent to Twin Stream. The Ben Ohau Range borders the western side of the lake, and a number of mountainous ranges (Hooker, Sierra, Liebig and Gammack) encircle the northern end along the outskirts of the Tasman River. The western and northern ranges are classified as part of the Rakaia terrane, which is a member of the Torlesse composite terrane. The dominant rock type in the catchment is quartzofeldspathic indurated sandstone (greywacke) interbedded with mudstone (argillite) of Permian-Triassic age (213 – 286 mya) (Cox and Barrell, 2007). Irwin (1978) states that the Pukaki catchment, as well as the Ohau catchment, also includes Haast Schists. Sequential stages of uplift in this area have been counteracted with ongoing erosion from glacial and fluvioglacial activity. The area has been subject to five glacial advances during the Late Pleistocene - Holocene. They are, from oldest to youngest: the Wolds (>100,000 B.P.), Balmoral (>35,000 B.P.), Mount John (22,000 - 17,000 B.P.), Tekapo (14,000 B.P.) and Birch Hill (11,900 – 9520 B.P.) glacial advances (Read, 1976; Porter, 1975). These glaciations role in erosion has resulted in distinctive landforms in low lying areas. Terminal moraines have formed on the plains on the eastern and southern shores.

Adams (1974) reported the Mackenzie Basin as an area of low natural seismicity. The Ostler Fault Zone, to the west of Lake Pukaki, has not been active within the last 3,000 y (Read, 1976). The Irishman Creek Fault is another active fault located to the east of Lake Pukaki. The most recent earthquake for the Alpine Fault, located approximately 45 km north of Lake Pukaki, occurred in A.D. 1717 and the rupture extended over a distance of 375 km (Wells *et al*. 1999). Earthquake recurrence for the Alpine Fault varies from ~100 y (years) to at least 280 y, but averages ~200 y, which is the lapsed time since the most recent rupture. The identification of fault zones and potential seismic activity is of importance concerning lakeshore management. This is because seismically triggered submarine landslides are capable of creating internal waves or possibly a tsunami within a lake. The size of the wave produced is dependent on the amount of material displaced, especially because of the steep slopes and deep water.

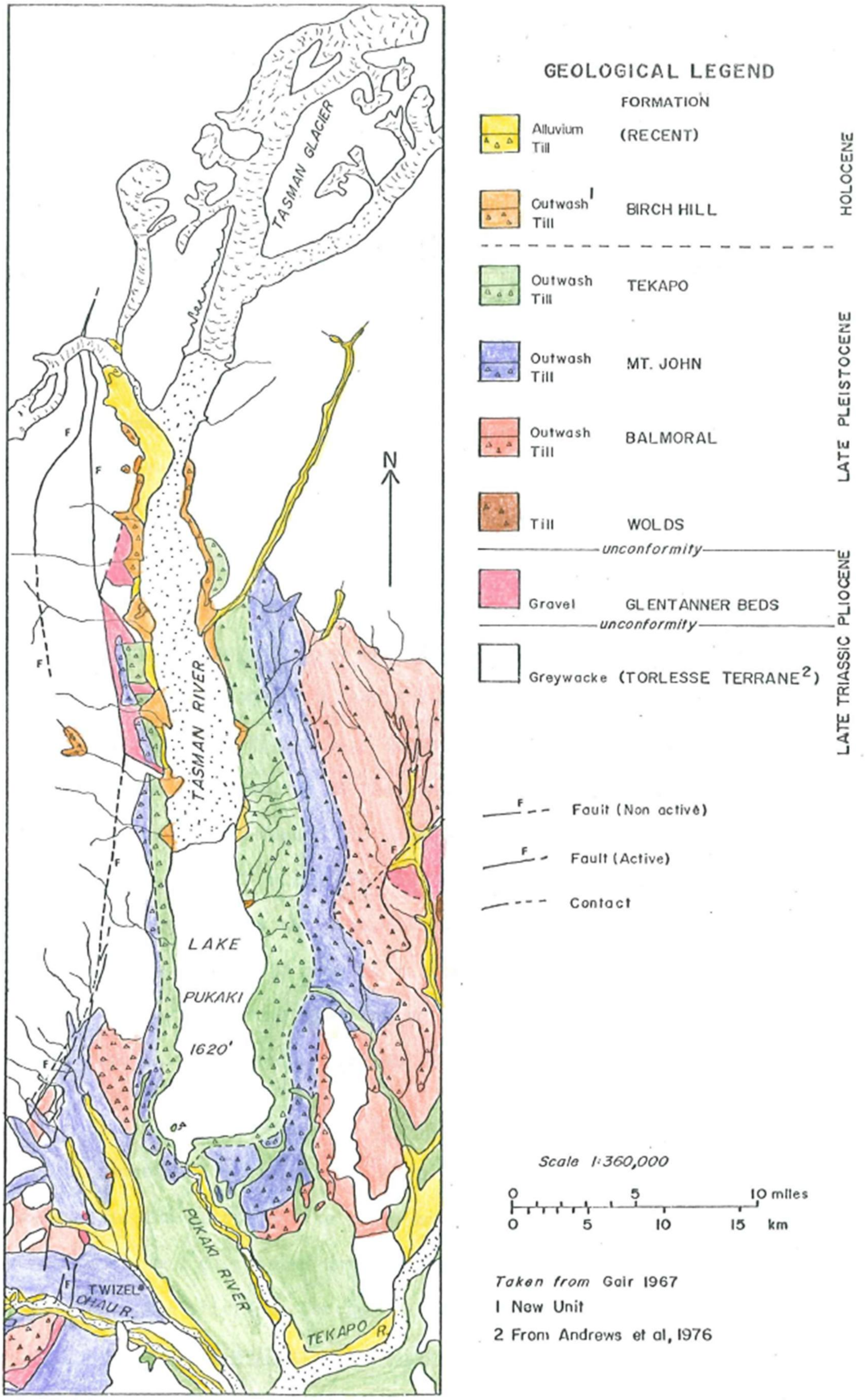


Figure 1.3: Regional geology surrounding Lake Pukaki. From Bunting (1977, Figure 5, p 22).

1.3.3 Shoreline Sediments

Barrell and Cox (2003) described the shorelines and clifflines of Lake Pukaki as very young, with the highest shoreline being 55 m above the original lake level. The current shoreline has developed owing to thirty three years of shoreline processes under a widely fluctuating water level. The western shoreline is characteristic of steep, cohesive, over-consolidated bluffs, especially in the southwest and further north. Comparatively, the southern and eastern shorelines portray a gentler topography. However, bluffs are localised in the southeast and north of the Braemar Road intersection (Figure 1.4).

The shoreline of Lake Pukaki is comprised of various deposits of till, alluvial outwash gravels, ice contact deposits and fluvial-glacial outwash (Bunting, 1977). A thin layer of glacial material has been deposited along the flanks of the Ben Ohau range. Towards the east and the south, extensive terminal moraine landforms exist. These have developed on a thin deposit of till that overlies thick, previously accumulated, outwash aggradation gravels (Barrell and Cox, 2003). During the Holocene, a number of processes have altered the landscape since deglaciation, this includes: fluvial activity, freeze-thaw and aeolian processes (Allan, 1991).

The fluvial system contributes considerable loads of suspended sediments, sands, gravels and boulders into Lake Pukaki. The Tasman and Jollie Rivers are the biggest contributors in terms of sediment input into the lake. The Jollie River flows into the Tasman River about 3 km upstream of the river mouth. Kirk (1988) predicted the total annual volume of sediment delivered to Lake Pukaki by the Tasman River is at maximum equal to 1,000,000 m^3. High sedimentation rates along the Tasman delta front can lead to submarine slumping, which can be seismically triggered, as explained earlier. Pickrill and Irwin (1983) estimated that more than 55% of the sediment entering the lake via the Tasman River is incorporated into delta building, with 40% being deposited in the basin and only 5% being deposited on the steep lateral slopes. They also indicate that clear evidence of submarine slumping has existed in the past. There are several smaller streams and creeks that also contribute a significant amount towards the sediment budget.

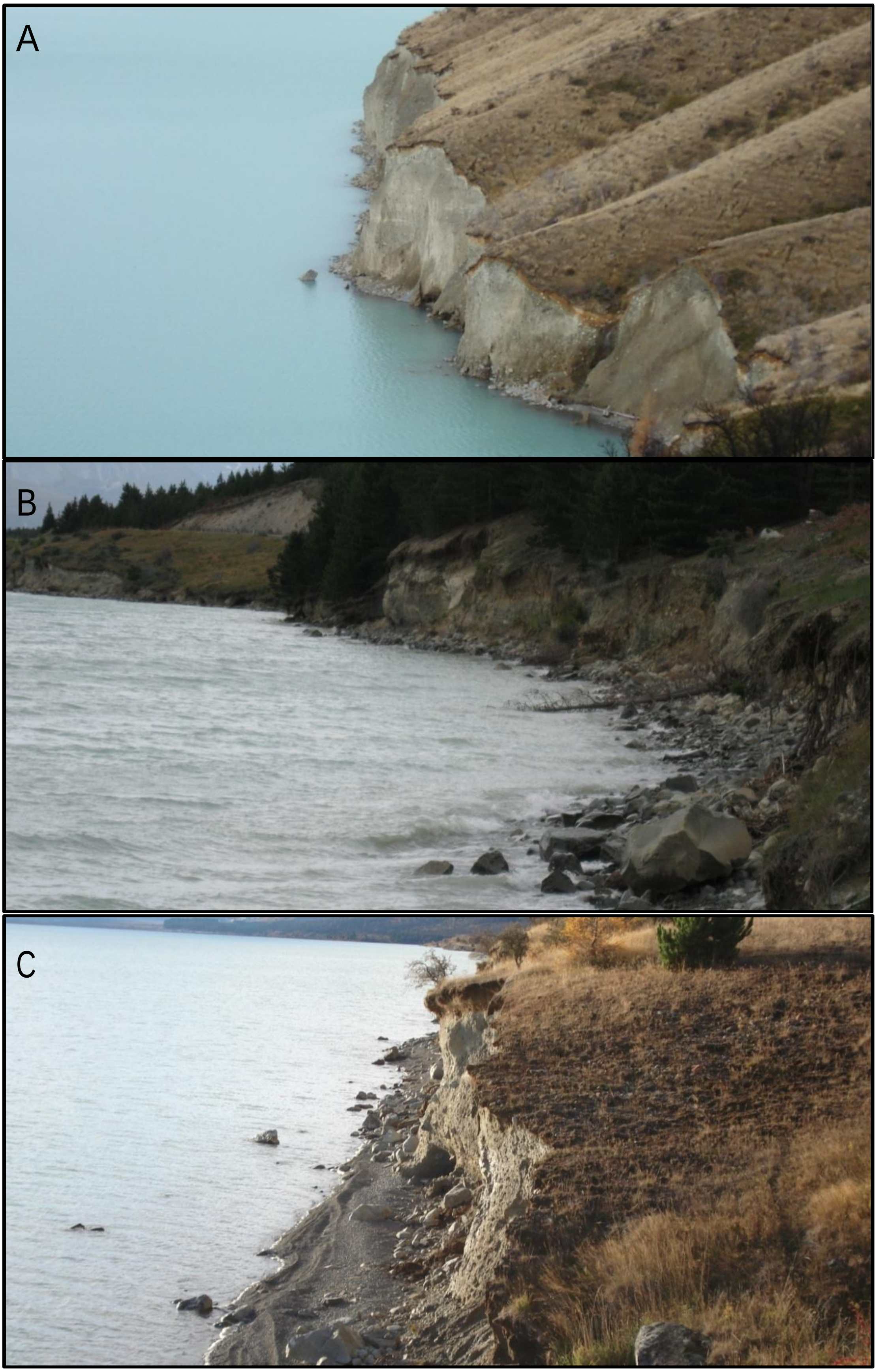


Figure 1.4: Steep bluffs on: A) the western shore, along the northern edge of the Whale Stream alluvial fan; B) the southeast corner, 2 km northeast of the Hayman Rd/SH 8 intersection; and, C) the eastern shore, 3 km north of the Braemar Road junction.

The majority of these originate within the Ben Ohau range on the western shore, for example: Twin, Whale, Jacks and Boundary Stream. Aeolian processes have also deposited immense sheets of loess (silt), with depths around the lake averaging 15 to 60 cm thick (Bunting, 1977).

Tekapo Till is the most widely distributed deposit about the shores of Lake Pukaki, and is affiliated with the Tekapo advance. It comprises light grey, homogenous, unweathered silt fine to coarse sand and fine to coarse gravel with rare cobbles, rare boulders and very rare clay (Bunting, 1977). Wave induced erosion of the till backshore has caused fine sediments to be released and dispersed into the deeper parts of the lake, while the less abundant coarse portions have been retained to form a fronting beach composed of coarse sand, gravels through cobbles, together with the occasional boulders. The coarser deposits also act as a natural armouring of the nearshore and foreshore.

1.3.4 Climate

Various aspects of climate play important roles in terms of the weathering and erosion processes operating in alpine lakes. The most significant process concerns the wind conditions because of their role in wave and current generation. Winds are channelled down the north-south, long axis of the lake, due to the steep narrow topography. Strong north-westerly/northerly winds can create considerable waves that reach the southern shore. The wind-wave regime is in more detail outlined and discussed in Chapter 4.

Rainfall and temperature are also important factors influencing lakeshore weathering and erosion processes. Figure 1.5 presents a map of precipitation isohyets (contours of total annual precipitation) from which it is noted that values increase exponentially northwards. Values range from 652 mm (+/- 50 mm) at the outlet of Lake Pukaki, to 10000 mm (+/- 2400 mm) towards the furthest point in the northwest (Kerr, 2009). Seasonal variation of rainfall is apparent in the north, near Mount Cook Village, with a summer maximum (4000 + mm) and a winter minimum (2000 mm). Comparatively, seasonality is not as evident in the south (1500 – 500 mm) (Kirk, 1988). Bunting (1977) accounted that the lower rainfall recorded in the north during the winter months is part due to precipitation in the form of snow.

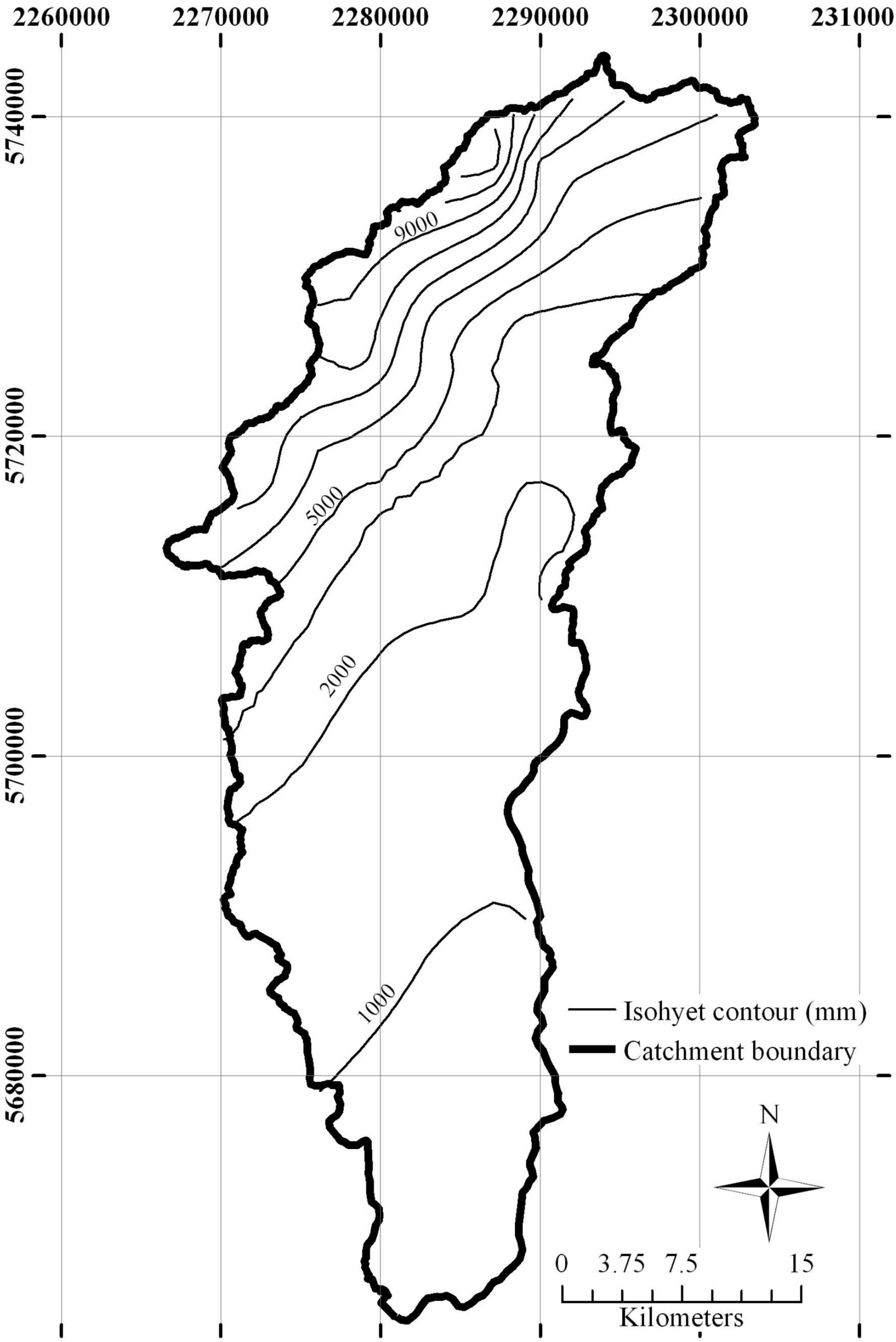


Figure 1.5: Estimated average annual precipitation distribution in the Lake Pukaki catchment, contours showing total annual precipitation. After Kerr (2009, Figure 3-9, p 99).

Air temperatures within the Pukaki valley also display a seasonal variation. Average daily temperatures can exceed 20°C in the summer and fall below 0°C in the winter (Kirk, 1988). Allan (1991) explains the importance of using air-water temperature differentials as a variable for wave prediction calculations. This will be elaborated on in Chapter 4.

1.4 Thesis Aims, Approach and Structure

The purpose of this thesis is to investigate the current lakeshore management strategies put in place to combat the extensive shore-wide erosion evident at Lake Pukaki. This involves assessing the practicality and performance of existing structures and their role in stabilising the shoreline. Knowledge of the nearshore process environment is required to understand the potential energy to cause erosion and the threat it poses to the road network surrounding Lake Pukaki's shoreline. This thesis therefore has five underlying aims:

1. To investigate the wind system around Lake Pukaki as wind-driven wave action is the most significant natural erosive force acting on the shoreline.
2. To measure wave characteristics using accurate wave recording devices, during high-magnitude storm events.
3. To correlate measured wave statistics against modelled wave statistics to ascertain the accuracy of estimating wave characteristics along the shoreline.
4. To examine the frequency and occurrence of high magnitude storm events coinciding with high lake levels.
5. To assess the current shoreline protection structures used to combat the ongoing erosion along the shoreline of Lake Pukaki.
6. To identify the controlling factors in determining the success or failure of different types of shore protection around the lake.

To achieve the above aims, this study applies known methods and techniques that have been successfully used in previous lacustrine and coastal environmental studies. New methods are introduced into the lacustrine setting and prior methods are improved upon.

This thesis is structured in a way to accommodate the principle aims mentioned above as follows.

Chapter 2 serves as a background chapter reviewing the pertinent material concerning lacustrine hydrodynamics. Concepts regarding wind waves and nearshore processes for alpine lakes are introduced. Lake level data is presented, and the effects that lake level fluctuations have of nearshore processes are examined. The morphology of alpine lake environments is discussed with the assistance of surveyed beach profiles.

Chapter 3 offers insight to the nearshore wave processes operating at Lake Pukaki. Allan (1991) explains that there is a paucity of accurate wave recording devices within lacustrine environments. Wave recording devices are introduced and their implementation for use in alpine lakes is evaluated. The results for measured deepwater wave statistics are presented and discussed.

The wind regime of Lake Pukaki is examined in Chapter 4, through the establishment of two automatic weather in close proximity to Lake Pukaki. This work is collated with wind data from previous studies stations installed on the southern and eastern shores. Comparisons are made with CliFlo weather stations on Lake Pukaki by Kirk (1988) and Allan (1991). Wave forecasting and hindcasting techniques will be implemented to predict wave statistics along the shoreline. Measured wave statistics at Lake Pukaki will be correlated with predicted wave statistics to test the model's ability at predicting wave statistics along the shoreline of Lake Pukaki.

The frequency and occurrence of high magnitude storm events coinciding with high lake levels is investigated in Chapter 5. Modelled waves statistics during extreme conditions will be presented and discussed. In terms of shoreline management, being able to identify the maximum elevation waves are able to act on the shore profile leads to a better understanding of design specifications required for coastal protection structures.

A background to erosion control structures in terms of their purpose and available options is introduced in Chapter 6. The history of shoreline mitigation and protection at Lake Pukaki is summarised. The quantitative and qualitative methodology used to assess the performance of the shore protection structures, surrounding the shores of Lake Pukaki, is described. The associated findings are then presented and discussed.

Chapter 7 serves as a discussion chapter. The suitability and performance of the erosion control structures in place will be justified. The environmental factors responsible for accelerated erosion rates along the shoreline, possibly leading to structure failure, are identified. The geological character of the backshore, including its resistance and stability, and the wave characteristics at particular sites, in terms of potential energy, will be assessed and determined. Recommendations for achieving erosion control will be outlined, which may involve the relocation of assets. The methodology is reflected upon, and possible discrepancies are identified.

Lastly, Chapter 8 presents the major conclusions for this study and provides recommendations for future research in lake environments.

Chapter 2

ALPINE LAKE PROCESSES AND MORPHOLOGY

2.1 Introduction

This chapter serves as a background chapter regarding alpine lake hydrodynamics and morphodynamics, which consists of three parts. First, concepts concerning wind waves and nearshore processes for alpine lakes are introduced. Second, annual lake level data for Lake Pukaki is presented, and the effects that lake level elevation changes have on nearshore processes are evaluated. Lastly, the morphology of alpine lake environments is discussed with the aid of surveyed beach profiles.

A large portion of this thesis assesses the measured and modelled wave environment at Lake Pukaki. Therefore, a review of the pertinent material concerning lacustrine hydrodynamics is required so that the reader is able to understand the dynamics of nearshore processes operating in an alpine lake setting. Knowledge of waves and the forces they generate is essential for the design of coastal projects since they are the major elements determining the geometry of beaches, the planning and design of shore protection structures and other coastal works (Demirbilek and Vincent, 2002).

The development of alpine lake processes resulting from strong northerly/north-westerly airflows has important implications on lakeshore modification (Allan, 1991). The orientation of the shoreline in relation to the dominant direction of wave approach is a key factor influencing the beach profile geometry. Knowing the elevation range that lake levels are able to fluctuate can determine the 'vertical window' for which wave energy is able to act and sub-sequentially modify the shoreline.

2.2 Lakeshore Processes

This section briefly introduces the processes active on alpine lakes. Concepts discussed are: the development of wind waves and their transition from deep to shallow water; occurrence of nearshore currents; and the mechanisms of sediment entrainment and transport.

2.2.1 Lake Wind Waves

The major generating force for waves is the wind acting on the air-water interface (Demirbilek and Vincent, 2002). The wind disturbs the equilibrium of the water surface via pressure and frictional forces, resulting in the formation of waves. The restoring force is provided by gravity as the waves break along the shoreline. As reported by Demirbilek and Vincent (2002, p 2):

> *"Wave energy forms beaches; sorts bottom sediments on the shore face; transports bottom materials onshore, offshore, and alongshore; and exerts forces upon coastal structures. A basic understanding of the fundamental physical processes in the generation and propagation of surface waves must precede any attempt to understand complex water motion in seas, lakes and waterways."*

Figure 2.1 demonstrates the transition of waves from deep water to shallow water. 'Deep water' is defined as water depths greater than half the wavelength and water particles move in closed circular orbits for each wave period. Wave action has meagre effects on the lakebed in 'deep water' and accordingly the lakebed does not exercise any influence on the wave form (Dawe, 2006). When waves propagate into 'intermediate water', where water depth is equal to half the wave length, the particle trajectories are compressed into ellipses and are able to interact with the lakebed. The influence of the lakebed leads to shoaling transformations and a progressive change of the wave form as the wave length decreases and the wave height grows. It is important to mention that during this process the wave period remains the same. As the waves continue to advance into shallower water the velocities of the wave motion are able to initiate sediment transport as the water particles demonstrate horizontal oscillatory movements (Dawe, 2006). 'Shallow water' is specified as water depths less than one twentieth of the wave length. Allan (1991) reported that the constraint exerted by water depth on wave generation will not occur for most of the glacial lakes of the South Island, principally because these lakes are very deep so that subaqueous influences are limited entirely to the nearshore zone. The steep gradient of these beaches allows waves to advance very close to the shoreline before breaking. This feature highlights the existence of a narrow surf zone on alpine lakes.

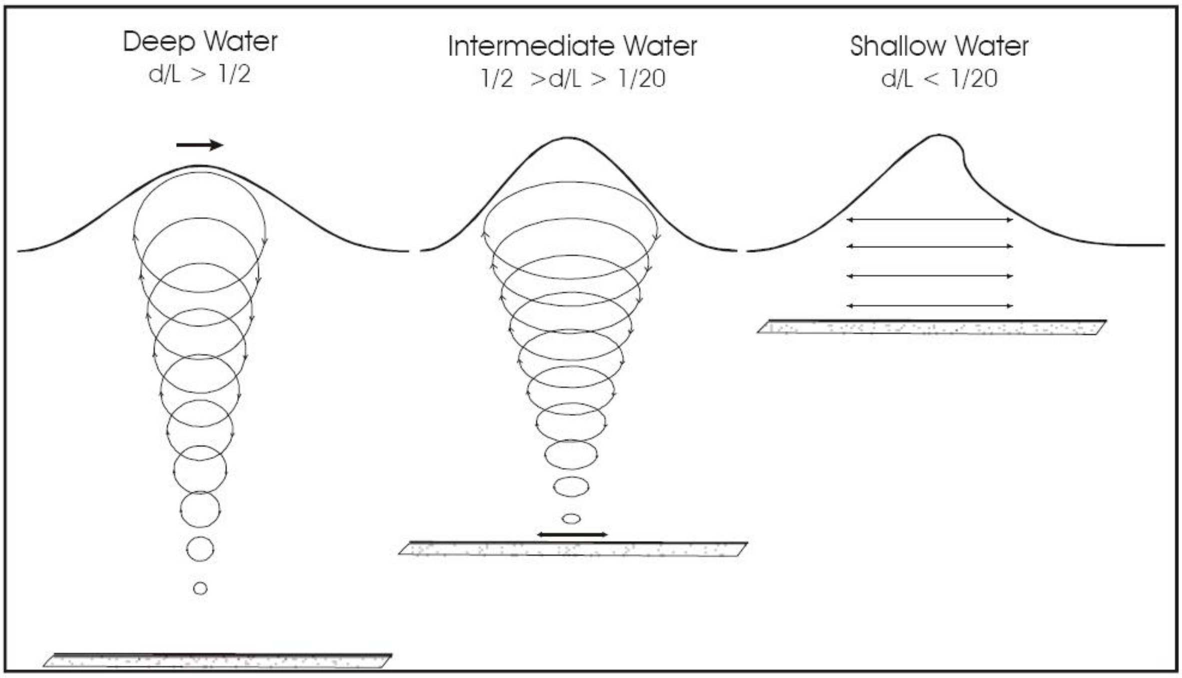


Figure 2.1: The transition of water orbital motions under a wave as it moves from deep to shallow water. In deep water the motions demonstrate closed circular orbits. As the wave propagates into intermediate water the motions become more elliptic as the wave interacts with the lakebed. In shallow water the motions become a series of horizontal oscillatory movements capable of transporting sediment. From Dawe (2006, p 5, Figure 1.2).

Deepwater linear wave theory can be applied for the majority of wave processes at Lake Pukaki, except for the occurrence of nearshore currents, which is discussed in Section 2.2.2. The generation of deepwater waves on lakes is dependant on three different factors:

$$H, T = f(U, F, D) \quad \text{Equation 2.1}$$

Where H is wave height, T is wave period, U is wind speed, F is the fetch length and D is the duration of the wind. As discussed in Section 1.2.1, the South Island alpine lakes are restricted by fetch, providing a limited area over which winds can develop and form waves. Winds are channelled within the confinement of glacial valleys on alpine lakes, typically resulting in a bi-directional wave climate. Prevailing wind flows that are aligned with the longest fetch have a significant influence on wave development, producing short steep waves that are largest at the downwind end of the fetch. The wind regime for Lake Pukaki is discussed and analysed in Section 4.3.

The growth of wind-generated waves on alpine lakes is not indefinite due to the restricted fetch lengths. The moment when waves stop growing is termed a fully developed sea condition or fully arisen sea (FAS) (Demirbilek and Vincent, 2002). During FAS wave height (H) and period (T) attain their maximum under a constant wind speed. FAS can be achieved within a few hours owing to the relatively short fetch lengths of alpine lakes (Kirk, 1988).

2.2.2 Nearshore Currents

Currents are important processes in the lakeshore environment as they enhance the initial sediment disruption of waves and contribute significantly to sediment transport (Kirk, 1988). There are several types of current systems that have been documented to occur on alpine lakes. They form in the nearshore zone, between the wave breaking zone and the shoreline and are dependant upon the beach morphology and the angle at which waves are directed towards the shore (Komar, 1998).

The first current system that can develop in the nearshore, and possibly the most important for alpine lakes, is a longshore current. Longshore currents are a major contributor in terms of sediment transport on alpine lakes. The presence of steep submarine slopes on alpine lakes causes minimal wave refraction to occur and therefore waves are able to approach large sections of the lakeshore at oblique angles (Allan, 1998). Longshore currents form due to the waves arriving at an oblique angle as part of the water motion is directed alongshore, which is illustrated in Figure 2.2A. The eastern and western shores of Lake Pukaki are subject to high rates of sediment transport due to their orientation relative to the direction of prominent wave propagation originating from the north and south. Figure 2.3 depicts the existence of a strong longshore current on the eastern shore at Lake Pukaki. For his study at Lake Coleridge, Dawe (2006) developed a formula to estimate longshore sediment transport, which is implemented for this study in Section 5.4.

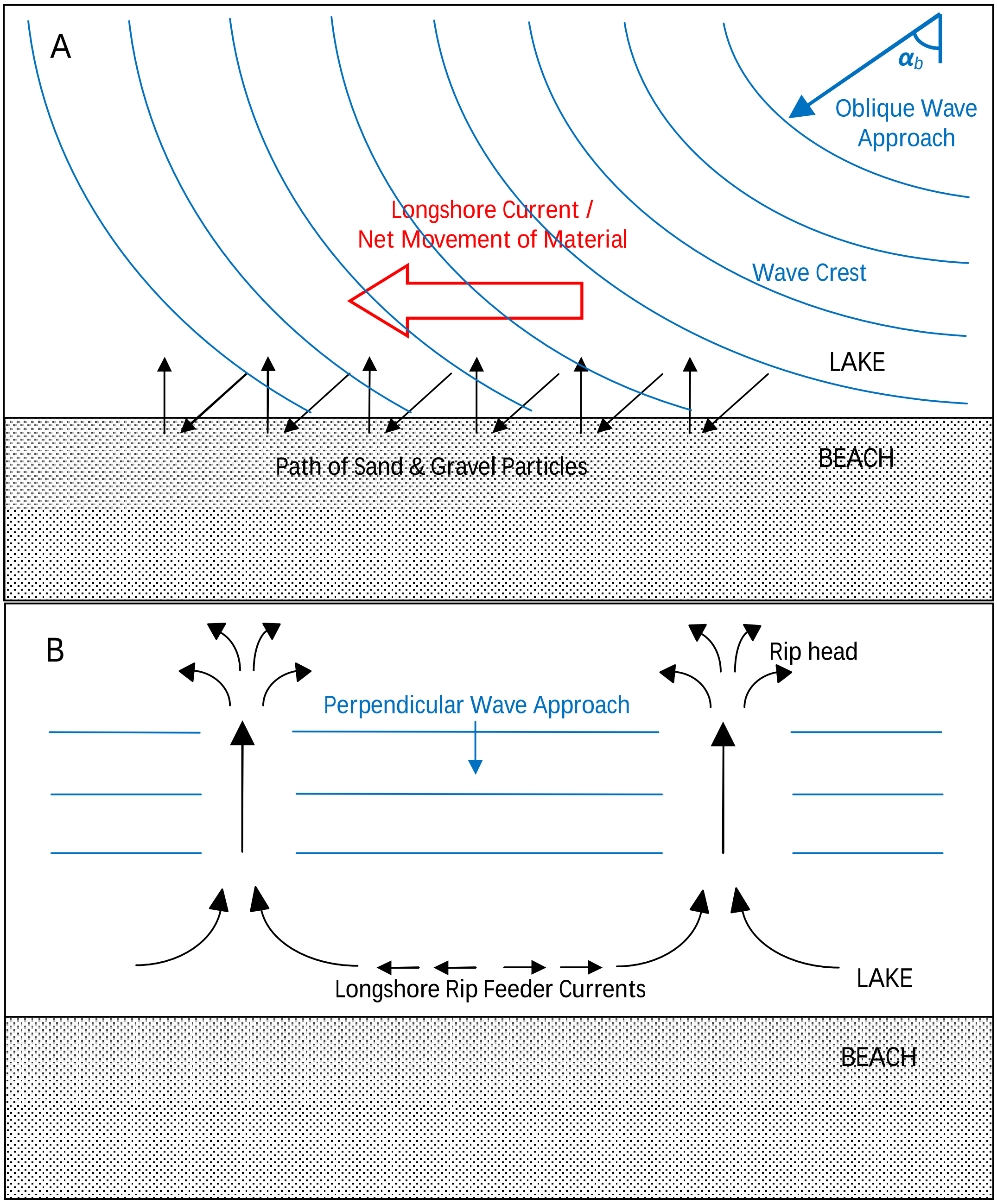


Figure 2.2: The formation of: A) a longshore current. The swash flows transport the sediments in a zigzag pattern, whereby the resultant direction is alongshore; and, B) a rip cell circulation. Shore-normal oscillatory flow moves sediments generally in an onshore/offshore direction.

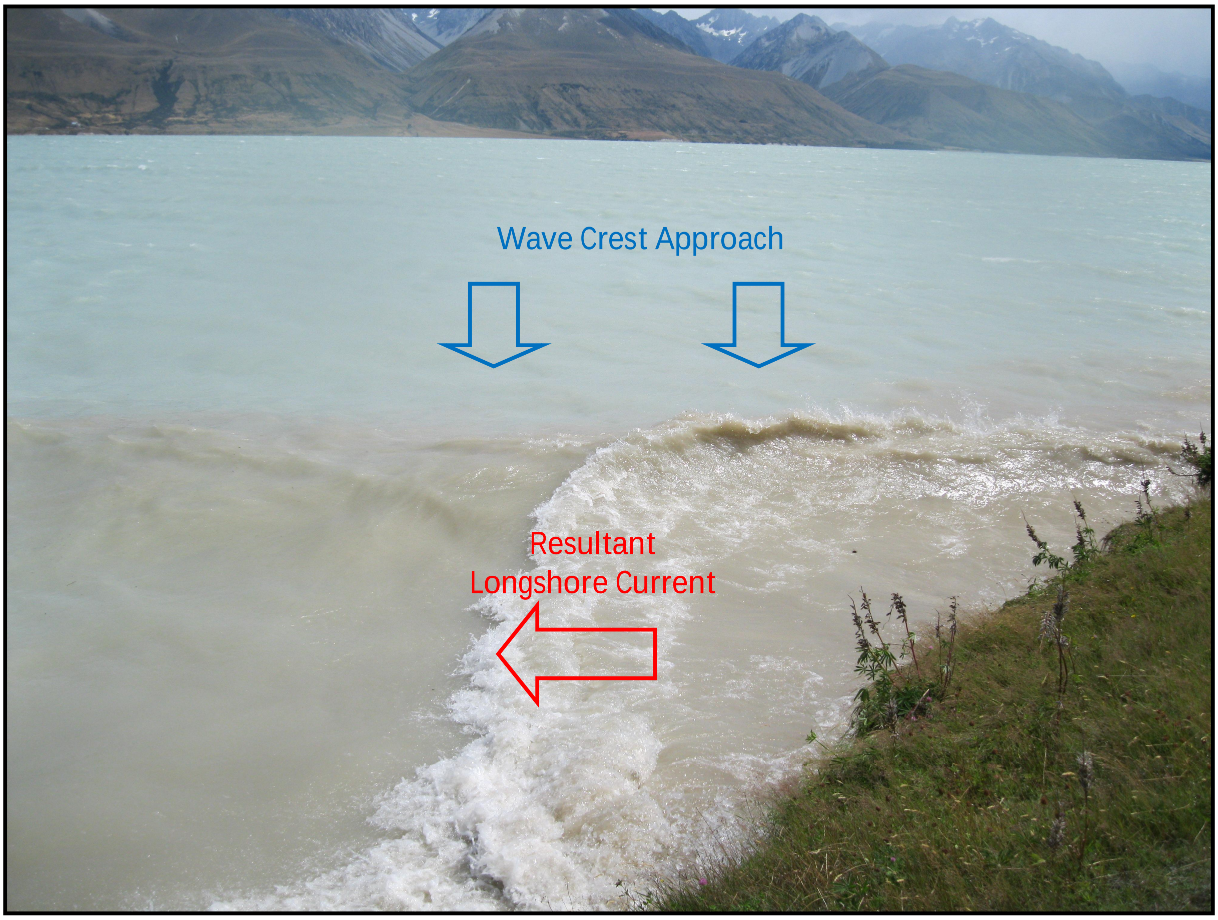


Figure 2.3: Longshore current development near Site 5. Wave energy reflecting off cliff face during a high lake level of 532.21 msl. Looking northwest. Photo taken on 6/02/11.

The second main current system is a rip cell circulation system, where the currents flow perpendicular to the shore. These currents are commonly known as rips and have the ability to recycle a great deal of sediment onshore and offshore (Dawe, 2006). They usually form along gently sloping shorelines and result when waves approach shore normal to a beach. A diagrammatic sketch of rip cell circulation system is presented in Figure 2.2B. The formation and existence of rip currents on alpine lakes have received limited attention. Allan (1991) reported the occurrence of rip currents at two sites along the southern shore at Lake Pukaki during strong northerly wind events. Upon visual observations he suggested that the rips were fed by both longshore and bottom return currents.

Wind induced surface currents and bottom currents are also considered important for sediment transport on alpine lakes. Strong winds blowing across lakes can lead to surface drift currents that have an average velocity of about 3% of the wind speed (Kirk, 1988). If the surface drift current is onshore, then it must be compensated by a bottom return current (undertow). Bottom currents may intensify the erosive nature of storm waves, entraining and transporting sediment offshore, where it is lost to the beach system (Kirk, 1988). At Lake Pukaki Allan (1991) measured maximum bottom currents that exceeded 0.8 ms^{-1}, while mean currents recorded were 0.378 ms^{-1}. He remarked that these currents are considerably stronger (40%) than those measured on the Great Lakes of North America.

2.2.3 Modes of Sediment Transport

The process of wave breaking exerts significant turbulence and pressure on the lakebed, resulting in the entrainment of sediment. It is within the swash zone, landward of the breaking wave, assisted by the occurrence of nearshore currents, both shore-normal and longshore, that sediment transport is most effective (Allan, 1991; 1998). The critical velocities needed to move sediments increases with the particle size. The velocities required to initiate motion are higher than those necessary to maintain the movement of sediments (Kirk, 1967).

Figure 2.4 shows the different types of sediment transport that occur in lacustrine, coastal and fluvial environments. Depending on their grain size sediments are transported as two different modes, as bed load or in suspension. Coarse fractions, such as cobbles and gravels, are transported in the bed load, in which the grains remain close to the bed. Bed load moves by sliding, rolling or saltation and progresses at a small fraction of the fluid velocity. Saltating particles will temporarily leave the bed and collide with other grains, setting them into motion and thus continuing the process (Dawe, 2006). Suspended sediment transports finer material, such as sands, silts, and clays, which is carried above the bed by turbulent eddies of water. Sediment is held within the lower to middle parts of the water column, and moves at a high percentage of the mean flow velocity. Suspension concentrations have been known to decrease with height above the bed (Rosati *et al.* 2002). The finest fraction, such as silt and clay, remain in suspension for a long duration and are often referred to as washload. It is ordinary for part of the load to be advected in suspension and a part transported in the bed load. Dawe (2006) noted that on mixed sand and gravel beaches, common on alpine lakes, it is the bed load transport that dominates.

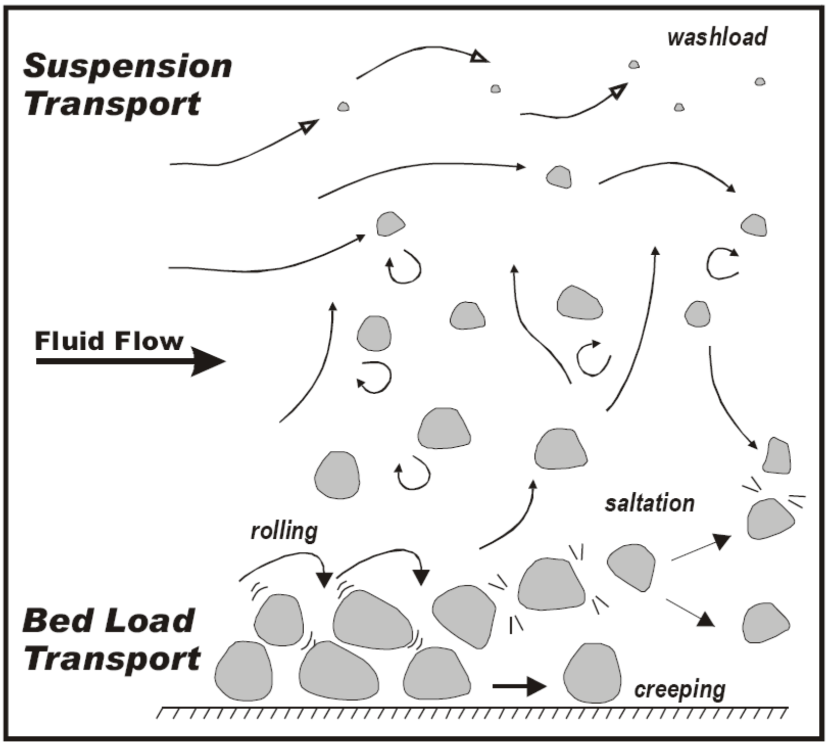


Figure 2.4: Different modes of sediment transport. Bed load transport moves coarser particles in close vicinity to the bottom, while suspension transport occurs when finer grains are held aloft in the fluid flow through turbulence. From Dawe (2006, p 7, Figure 1.4).

2.3 Lake Levels

2.3.1 Lake Levels: 2003 – 2011

Figure 2.5 summarises the lake level data, each year, from 1/1/03 to 6/2/11. The normal operating range for Lake Pukaki is between 518.2 and 532 msl. However the lake level has not dropped beneath 520 msl in the past eight years. A strong seasonal component is evident, with the maximum lake levels occurring during the summer and early autumn, and the minimum levels throughout late winter and early spring. This seasonal pattern occurs because the lake levels are regulated and artificially raised to create a buffer before the winter months when power generation is at its highest. Since 1/1/03 to 6/02/11 the upper end of the operating range of 532 m has been exceeded daily 8.14% of the time, whereas between 1/1/09 and 6/02/11 the 532 m mark was surpassed 23.96% of the time. This may be a result of operational and climatic variability.

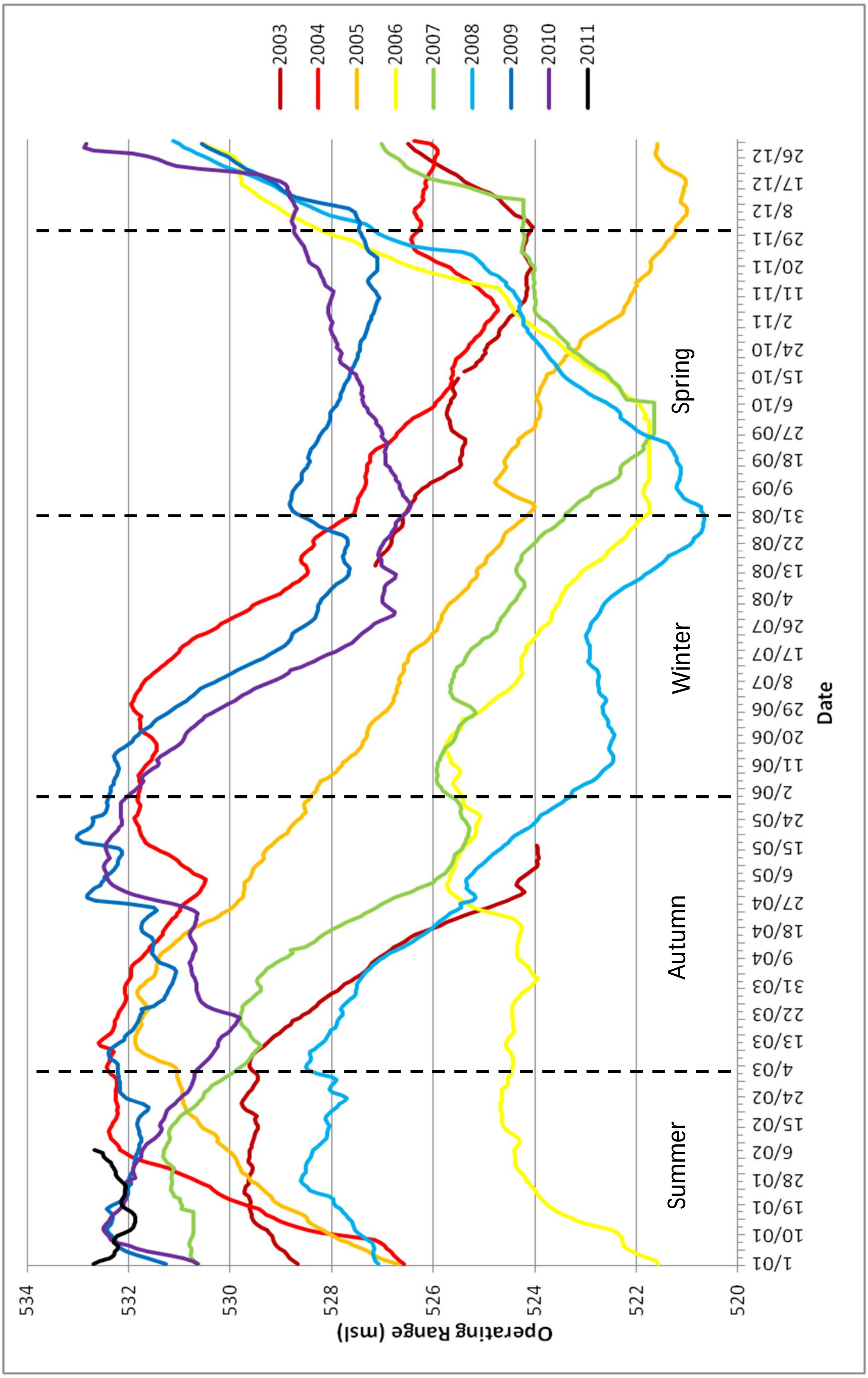


Figure 2.5: Lake level data for Lake Pukaki, from 1/1/03 to 6/2/11. Gaps within a year event represent a period when no data was obtained (total 3.07% no data).

Climatic variability, such as rainfall and runoff, from year to year can result in a variation away from the annual trend shown by the lake levels (Single and Bunting, 2003). Lake Pukaki's lake level also experiences short term movements in response to heavy rainfall.

The maximum lake levels during the summer period coincides with a greater magnitude of strong northwest/northerly winds resulting in the formation of highly erosional wind waves, that have had substantial impacts on the shoreline of Lake Pukaki (Allan, 1991), which is examined in Chapter 5. The role of lake levels and their effects on physical processes is explained in the following section.

2.3.2 Role of Lake Levels

Lake levels fluctuate naturally due to climatic inputs, such as rainfall and snowmelt. In the case of an artificially controlled lake, such as Lake Pukaki, the lake level endures a much greater range of variation of up to several metres. Lake level affects the physical processes acting within the lakeshore environment and is dependant on the elevation and frequency of the current water level. Lake levels determine the base level for which wave action is concentrated on the shoreline (Kirk, 1988, James *et al.* 2002) and the horizontal span of shoreline that energy is spread across. Figure 2.6 illustrates this concept.

James *et al.* (2002) note, high lake levels encourage shoreline erosion in two ways. Firstly, and importantly, high lake levels combined with strong winds and storm waves can cause a significant amount of erosion to take place along the shoreline. The toe section of a bluff, not usually subject to wave action, can be eroded, which can lead to instability of the slope and eventual failure as shown in Figure 2.7. Secondly, high water levels can raise the water table and saturate the beach sediments, causing finer particles to become mobile and be transported off shore. If the shoreline comprises of glacial till or other cohesive material, such as found on the shoreline of Lake Pukaki, it could be saturated for a long duration and susceptible to erosion by wave action or slumping if the lake level is dropped rapidly (James *et al.* 2002).

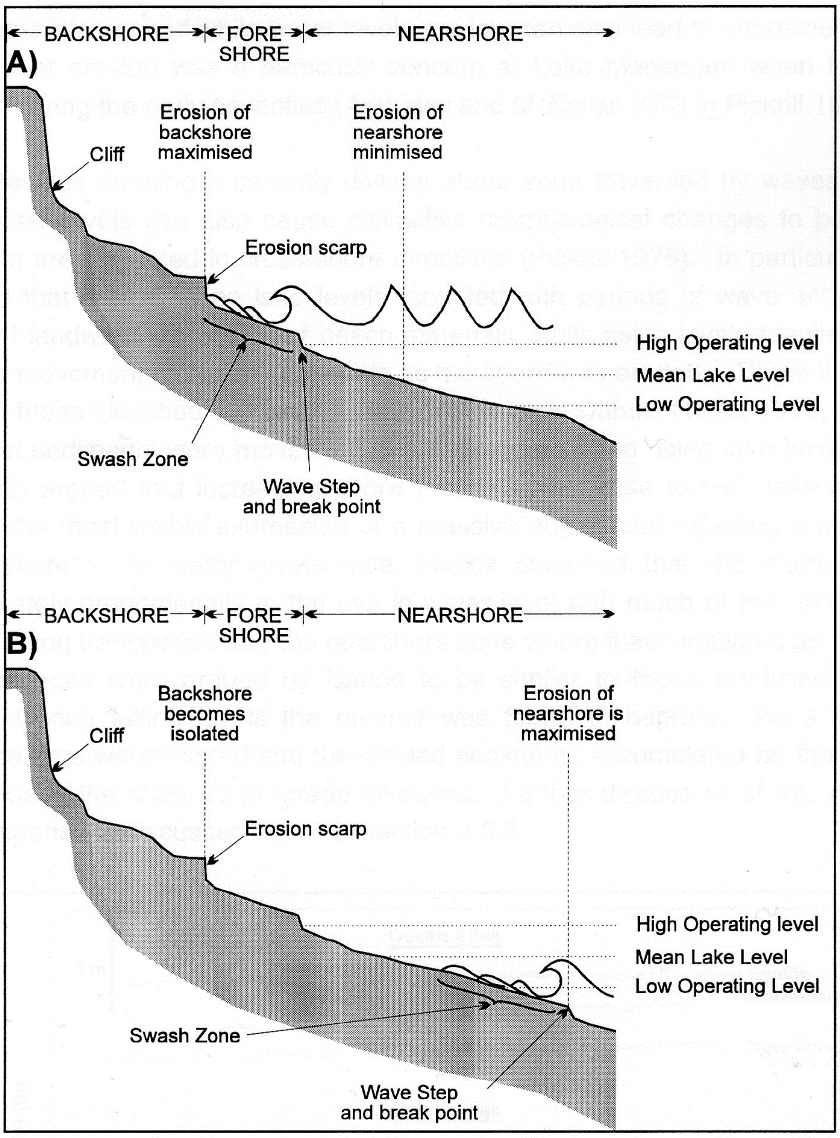


Figure 2.6: Effects of water level fluctuations coinciding with the occurrence of storm waves on a lakeshore: A) during high lake levels, erosion to the foreshore and backshore takes place with minimal interference to the nearshore shelf; and, B) during low lake levels, the backshore becomes isolated and erosion is concentrated on the nearshore shelf. From Allan (1998, Figure 2.8, p 41).

Knowledge of wind stress set up, wave set up and wave run up during short term storm events, along with the still water level, are important in determining the maximum elevation wave processes can act on the shoreline. Kirk (1988) evaluated that a combined effort of wind stress set up and wave set up raised Lake Pukaki more than 0.5 m, for wind speeds between 20 and 24 ms^{-1} from the northwest. These calculations are important, though often disregarded, for designing shore protection structures along lakeshores (James *et al.* 2002). Figures for these, including wave run up, are presented in the Section 5.2.

Not only do high water levels have an impact on lakeshore processes, but so do low water levels. Due to the generally narrow, shallow nearshore shelf on lakes, low water levels can focus wave energy on the upper edge of the offshore face. Steep storm waves in conjunction with bottom return currents can move sediment from the offshore face into the deeper water, resulting in a loss to the sediment budget (James *et al*. 2002). Removal of sediment from the nearshore shelf can also create instability within the submarine slope and possible slumping. Material can also be lost from the beach system when structures such as berms are abandoned at the limit of wave run up, during a subsiding water level. When the water level is situated at a specific elevation for a long time low frequency storm events can form steps across the beach profile. An example of this shore feature is shown in Figure 2.8.

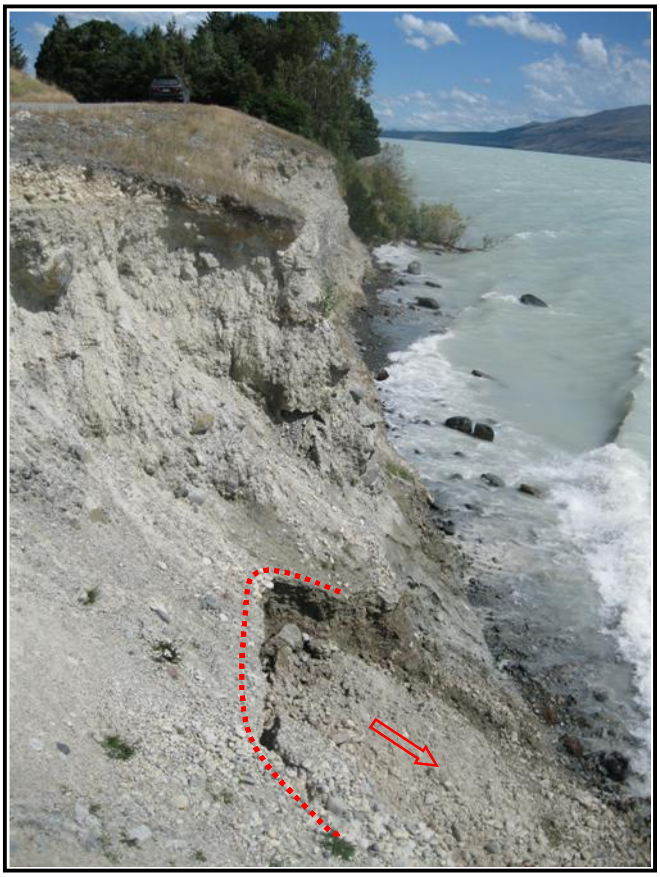


Figure 2.7: Slumping at the toe of a cliffed section, 300 m north of the Braemar/Hayman Rd intersection, on the eastern shore, looking south. High lake levels have concentrated wave action at the toe of the bluff and caused failure. The dashed red line defines the newly formed erosion scarp and the red arrow indicates the direction of movement. The top segment of the bluff has eroded back to ~1 m from the roads edge. Lake level was 532.509 msl. Photo taken on 6/2/11.

Figure 2.8: Step feature, ~0.5 m in height, adjacent Tasman Downs Homestead, on the eastern shore, looking north. Step structure extends ~100 m along the beach. Note the lack of fine sediment lakeward of the step and the abundance of gravels and boulders. This site was surveyed and its profile is analysed in Section 2.4.2. Lake level was 528.067 msl. Photo taken on 6/11/10.

2.4 Shoreline Morphology

Having reviewed the shoreline sediments at Lake Pukaki previously in Section 1.3.3, this section briefly discusses the beach morphology typical of alpine lake environments and that of Lake Pukaki. Several sites along the Lake Pukaki shoreline were surveyed for the purposes of determining the beach slope angle, for calculating maximum wave run-up heights ($R_{2\%}$), carried out in Section 5.2. These profiles have also been utilised to describe the shore morphology of Lake Pukaki.

Figure 2.9 shows the location of the twelve study sites used for this research. The locations of these sites were chosen based on field work and recent investigations being carried out in these areas.

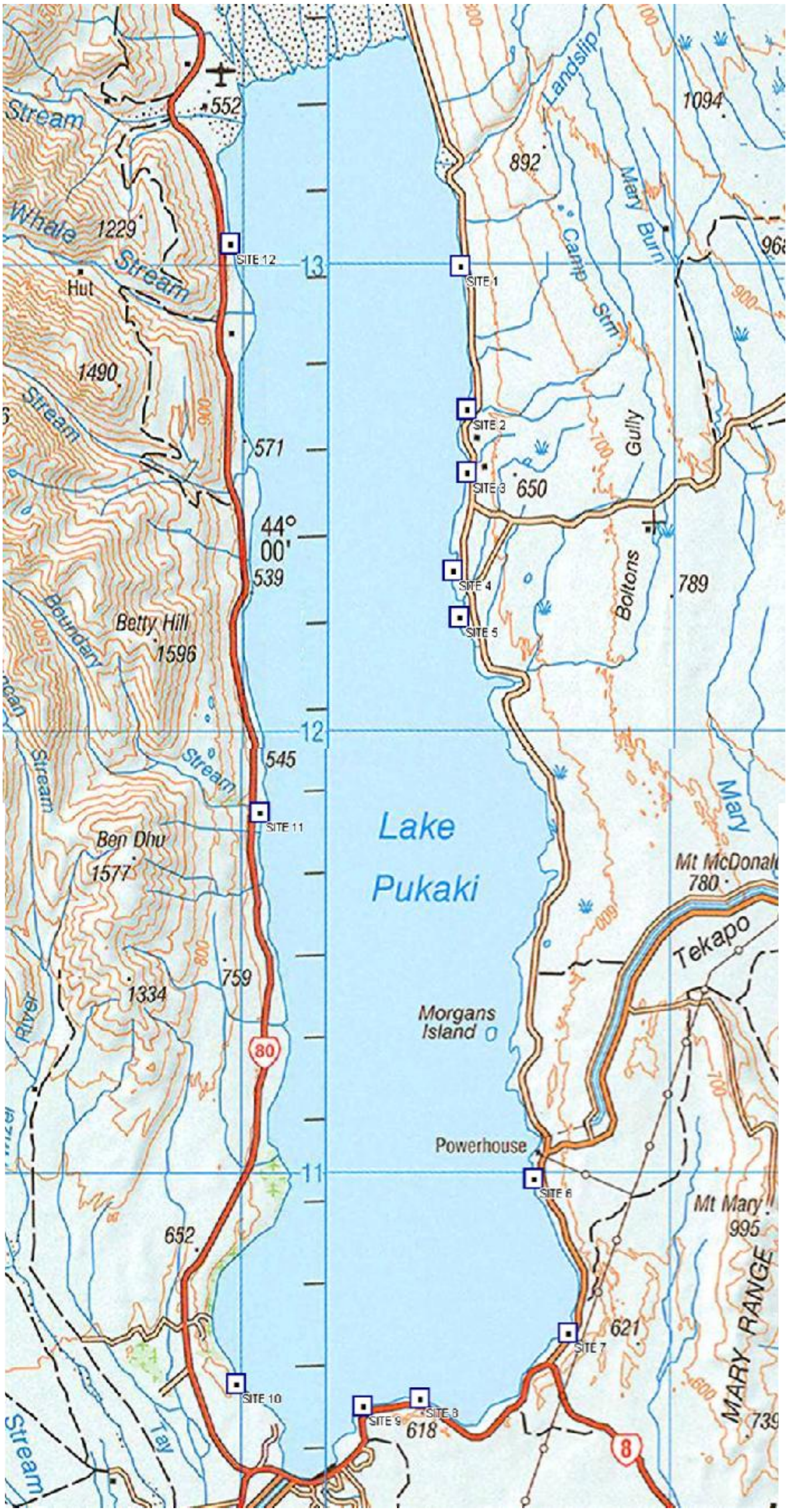


Figure 2.9: Location of the twelve study sites at Lake Pukaki (TopoMap, 2010).

2.4.1 Shoreline Topographical Surveys

Six shoreline profiles (Sites 2, 4, 5, 7, 8 and 11 of Figure 2.9) were surveyed around Lake Pukaki; one on the western shore, two on the southern shore and three along the eastern shore. Surveys were done on the 5/11/10 and 6/11/10 when water levels were low, which allowed for a larger section of the foreshore to be quantified. The coordinates were established by the use of a Garmin hand held GPS. This is accurate to +/- 5 m, which is sufficient to show the location of each site. The level control was established from the lake level from that day in terms of mean sea level. This was found on the Meridian Energy website. In the field a series of measurements were taken at the water level to give the correct levels for each site. Once the level and coordinate control is accomplished a topographic survey can be completed. To do this, traditional methods of surveying were used. This involved a Sokia set 3e, serial number: 30970, which is accurate to +/- 10 mm, shown in Figure 2.10. It required a surveyor behind the total station taking measurements to a prism pole, which the second person or chainman was walking around with. The topographic survey sites were roughly 50 m^2. Levels were attained every 10 m or change in grade or feature. The survey information was then downloaded into AutoCAD Civil 3d 2011 where a digital terrain, or contour model was created. Two long-sections, for each site, were produced from the contour model to show the beach profiles. Profiles are exaggerated two times in height when plotted. The individual shoreline profiles are assumed to represent the entire site from which they were located. Employing the use of a 3-D model would be more accurate but is not required for the purposes of this research.

2.4.2 Alpine Lake Beach Types and Morphology

The shape of a beach in profile is a function of inter-relationships between several important elements including waves, nearshore currents, varying lake levels, sediment size and the rundown of the sediment budget (Allan, 1998). During his research at Lakes Manapouri and Te Anau, Pickrill (1976) reported that mixed sand and gravel beaches on the South Island alpine lakes tended to develop a distinct three-part morphology, presented in Figure 2.11.

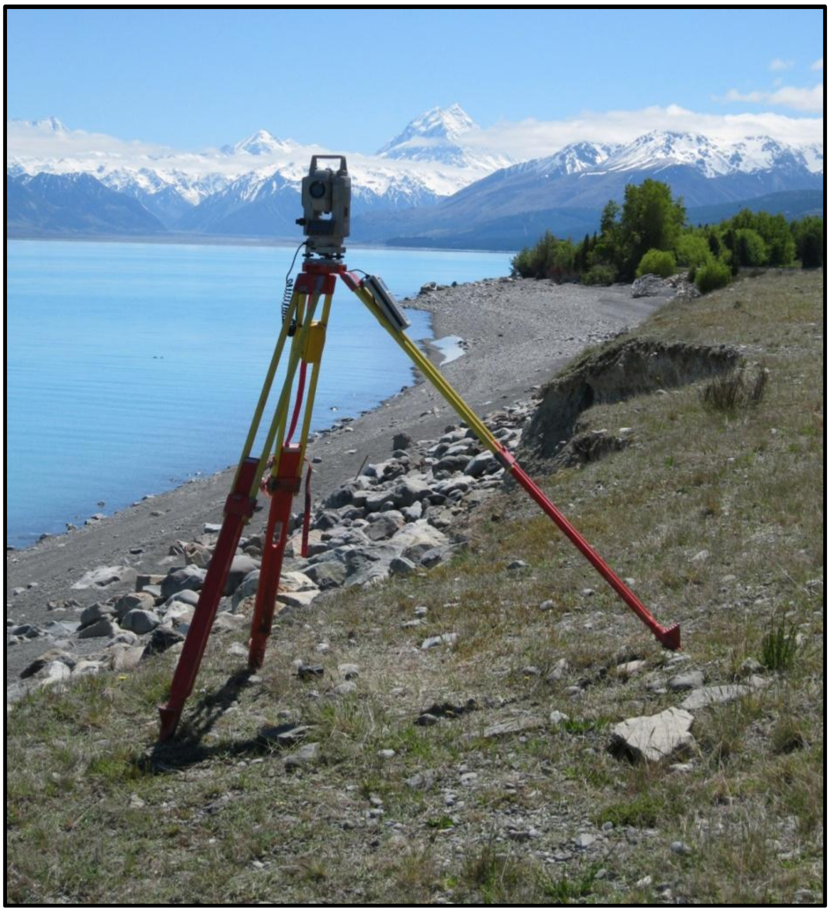


Figure 2.10: Sokia set 3e total station set up while surveying Site 4 on the eastern shore. Looking north. Photo taken on 6/11/10.

This included a moderately steep (7 - 8°) foreshore, a gently sloping (5 - 6°) nearshore shelf and a steep offshore face. The backshore can be considered as the fourth element in Figure 2.11 for Pickrill's (1976) alpine beach profile. According to Dawe (2006) most of the morphological changes occur in the foreshore, not in the nearshore as on sandy beaches. The width and depth of the nearshore zone controls the distance over which energy dissipation processes occur. Therefore, sites exposed to longer fetches, and subsequently higher wave energies, tend to develop wider, flatter nearshore shelves (Pickrill, 1976).

Pickrill (1976; 1985) identified four types of beaches on alpine lakes, which he classified as: pavement beaches, two types of mixed sand and gravel beaches and sandy beaches. The presence of sandy beaches on the South Island's alpine lakes is uncommon, especially at Lake Pukaki as none were observed in the field. The three other identified alpine beaches are evident at Lake Pukaki and their characteristics are described below.

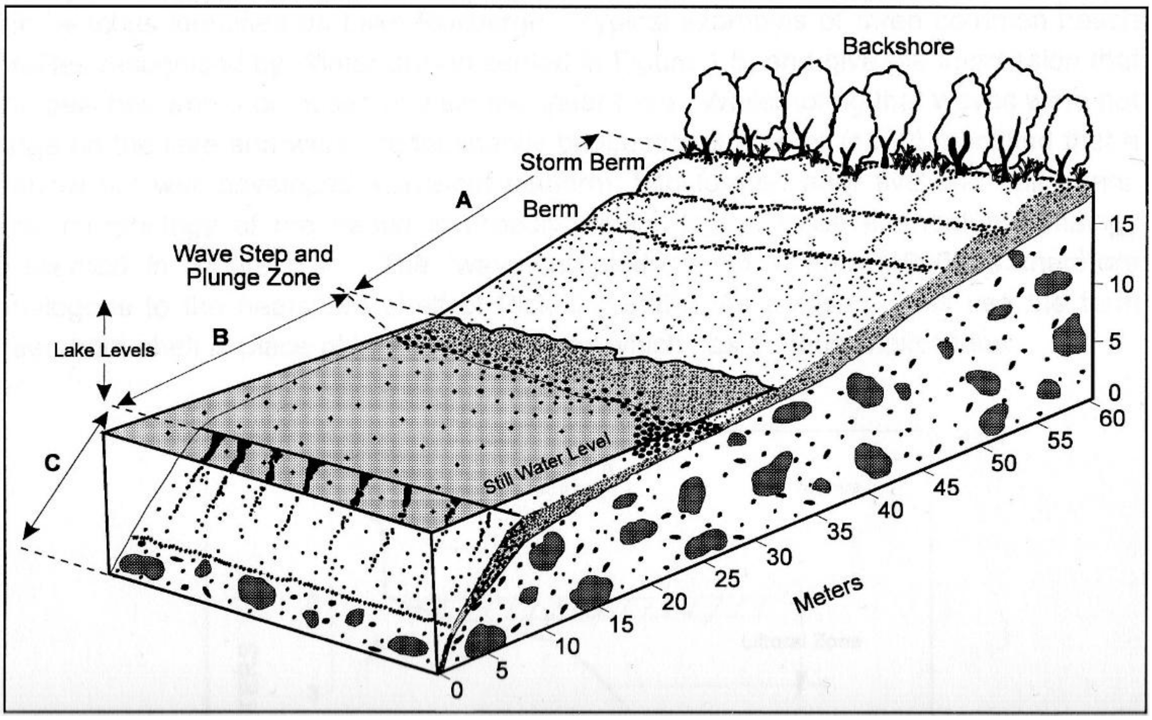


Figure 2.11: Typical New Zealand alpine beach profile with its three element morphology: A) the beach foreshore; B) the nearshore shelf; and, C) the offshore face. The backshore can be regarded as the fourth element. From Allan (1998, p 10, Figure 1.4).

The orientation of a lakeshore relative to the prevalent direction of wave propagation is a controlling aspect in determining the type of beach profile (Single and Kirk, 2003). With respect to Lake Pukaki, the long axis of the lake is aligned with the dominant north westerly/northerly wind flow. Beaches on the eastern and western shores are subject to an oblique wave approach angle and experience strong longshore transport. The associated shorelines are characterised as 'pavement beaches' (drift-aligned), as most of the finer sediment has been transported by the shore-parallel currents, leaving coarser gravels, and boulders behind (Figure 2.12). The remaining coarser material, also known as lag deposits, act as natural beach armouring and aid in maintaining the stability of the beach profile.

The second type of beach identified by Pickrill (1976; 1985) is a mixed sand and gravel beach containing predominantly coarser sediments. Site 4 (Figure 2.13), located on the eastern shore, is an example of this type of beach as it consists of a narrow steep foreshore (9°) and a beach step can develop during low energy conditions at the wave breaking point.

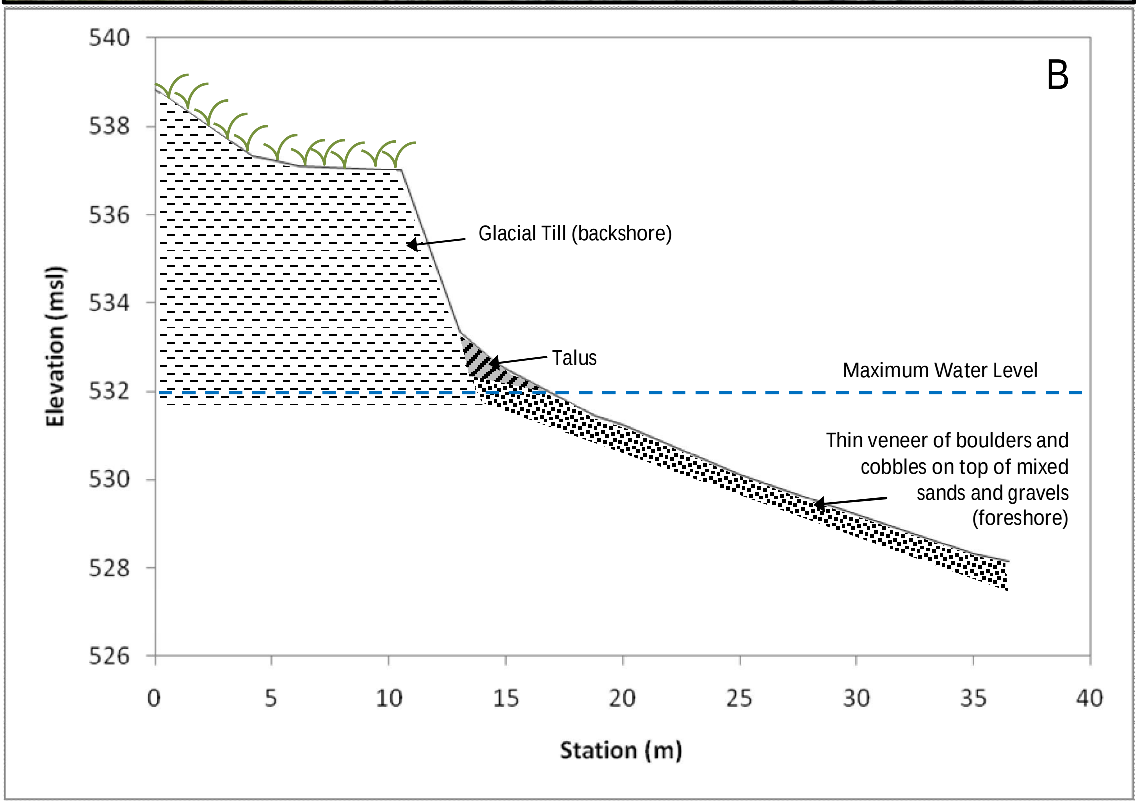


Figure 2.12: A) Site 11 on the western shore, looking north. Southern end of rock revetment near Boundary Stream. Typical 'pavement beach' subject to the high occurrence of longshore currents. Photo taken on 6/11/10. B) Longsection profile of Site 11A, two times vertical exaggeration. Talus accumulation on the lake side of the cliffed backshore acts as protection for any potential incoming wave energy directed at the base of the cliff.

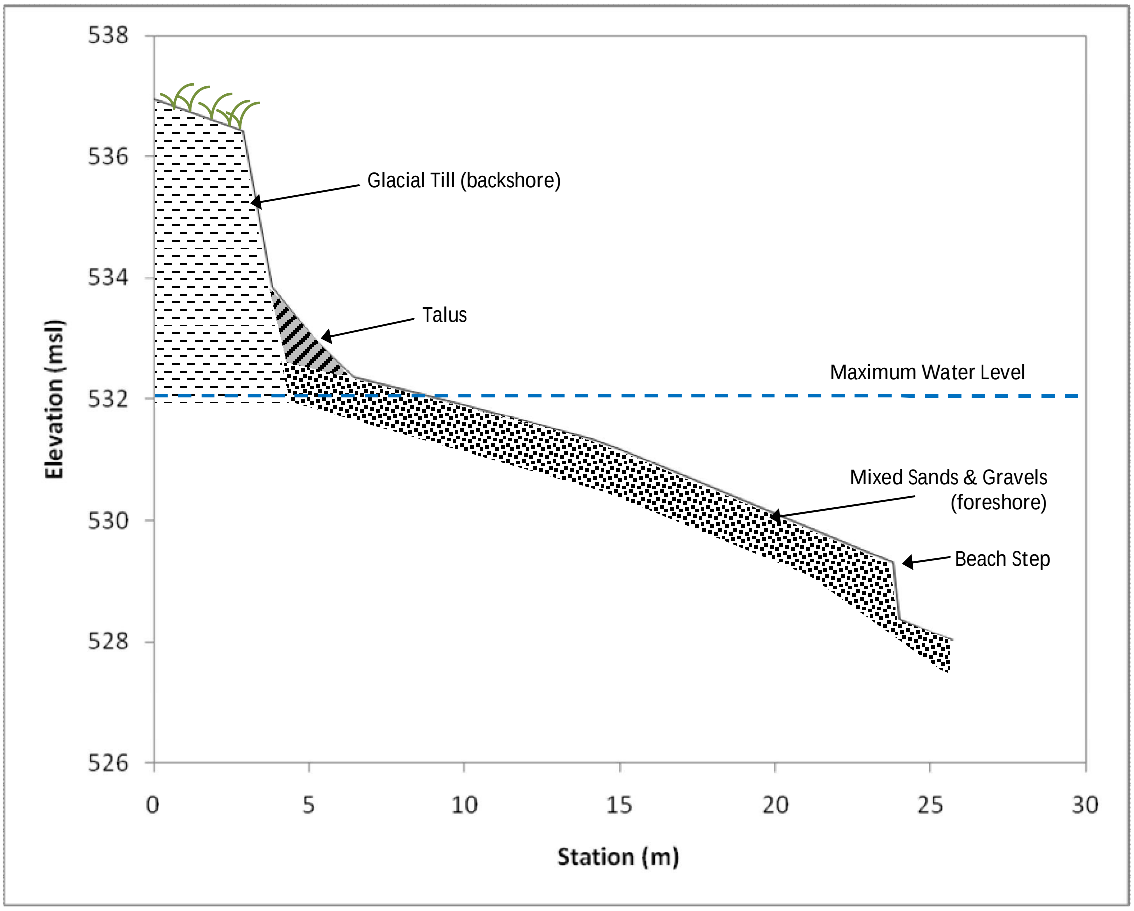


Figure 2.13: Longsection profile for Site 4A situated on the eastern shore, two times vertical exaggeration. The beach step at this site is also presented in Figure 2.8.

According to Pickrill (1976; 1985) these beaches are characteristically dominated by swash and backswash processes. This is not the case for Site 4 as longshore processes dominate due to its shore orientation receiving most wave energy at an oblique angle.

The southern shores receive the majority of wave energy perpendicular to the shoreline and are regarded as swash-aligned beaches. Considered as the third alpine beach type (Pickrill, 1976; 1985), these beaches are made of medium to coarse sands and finer gravels (Figure 2.14), and are generally flatter (7 - 9°) in comparison to the eastern and western shores (7 - 14°). Depositional ridges may form on these beaches due to the shore-normal wave approach. These features can become stranded at the upper limits of the foreshore when lake levels undergo a drop in elevation.

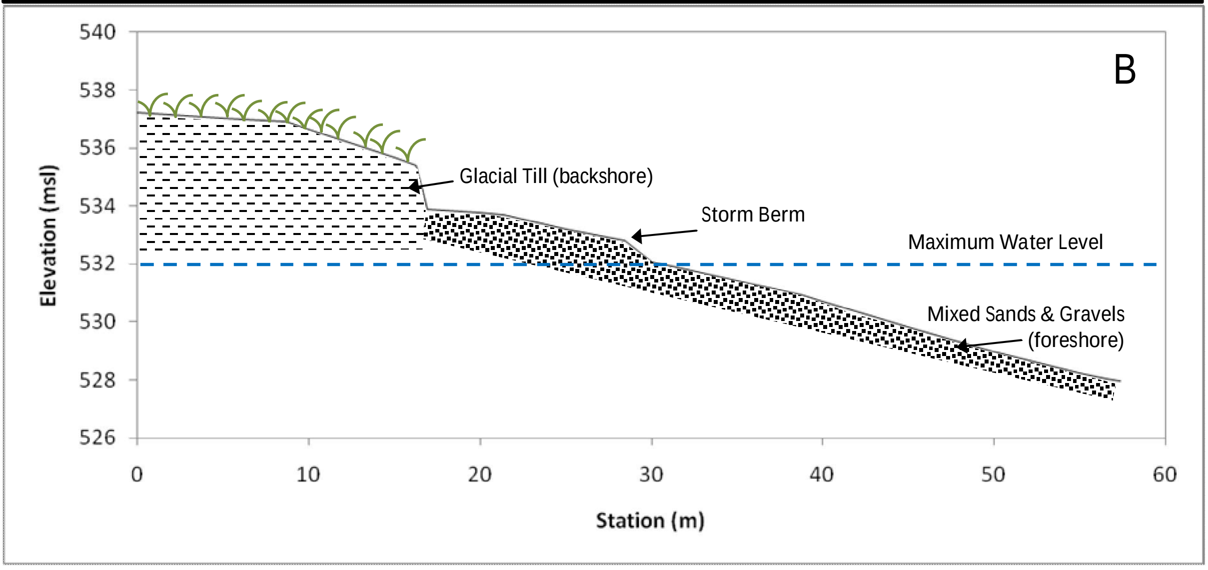


Figure 2.14: A) Site 7 on the southern shore, looking west. Characteristic swash-aligned beach composed of medium to coarse sands and finer gravels, and backed by a thin deposit of glacial till. B) Longsection profile of Site 7A, two times vertical exaggeration. The storm berm depicted on the longsection is evident in the photograph by a long stretch of driftwood, which has been deposited through swash processes during high lake levels. Photo taken on 5/11/10.

In contrast to the moderately steep beach profiles common along the eastern shore, the surveyed beach profile at Site 2 demonstrates a relatively flat profile (5 - 6°) similar to the beaches found on the southern shore. This site is exposed to waves produced during northerly wind events and experiences the majority of waves approaching the beach at a shore-normal direction. This explains the reasonably flat beach profile at this site as it is characteristic of a swash-aligned beach. Examined later in Section 5.2, the angle of the beach foreshore has a significant influence on the maximum wave run-up achieved.

2.5 Summary

This chapter has briefly discussed the hydrodynamics and morphodynamics of alpine lake environments, where a number of key features need to be re-emphasised. The development of wind waves on alpine lakes is dependant upon three factors: wind speed; wind duration; and the fetch length. The restricted fetch and channelisation of winds along glacial valleys produces short steep erosive waves. The steep subaqueous profile and the beach foreshore on the South Island's alpine lakes allow waves to propagate in close proximity to the shoreline before breaking. Therefore, deepwater (linear) wave theory can be applied to alpine lake settings as the wave form undergoes minor alteration in terms of shoaling and refraction in the nearshore zone.

It is within the nearshore zone that longshore currents are dominant. The eastern and western shores at Lake Pukaki are subject to strong longshore currents due to the shoreline orientation relative to the approach of wave propagation directed along the north-south axis of the lake. This has had a major influence on the morphology of the eastern and western shores, which consist of pavement and mixed sand and gravel beaches. These are considerably steeper that the wider, flatter beaches on the southern shore, which experience the majority of wave energy at a shore normal approach.

The lake level elevation has been identified as a controlling factor for the level at which wave action is concentrated on the beach profile. It is the coincidence of storm conditions with high lake levels that can lead to significant erosion along the backshore. This concept is examined later in Section 5.2, 5.3 and 5.4 concerning maximum wave runup values, wave power and longshore sediment transport rates respectively.

The following chapter examines the practicality and reliability of measuring waves via instrumentation at Lake Pukaki. The deep water wave statistics are then correlated with modelled wave characteristics estimated from local wind records in Chapter 4.

Chapter 3

QUANTIFICATION OF NEARSHORE PROCESSES

3.1 Introduction

There have been few measurements of waves on New Zealand lakes. The only documented wave recordings on the South Island alpine lakes are from Lakes Manapouri and Te Anau (Pickrill, 1976), Lake Dunstan (Allan, 1998; Allan and Kirk, 2000) and Lake Coleridge (Dawe, 2006). The paucity of wave recording has resulted in a major reliance on the use of wave hindcasting techniques to quantify nearshore processes. However, the majority of cases where wave hincasting models are utilised are seldom correlated to site conditions.

This chapter provides insight to the nearshore processes operating at Lake Pukaki, with emphasis on deepwater wave statistics. Although Allan (1991) made wave height observations, this study marks the first time for which instrumented wave measurement has been implemented at this lake. An XR-620 pressure sensor and a WG-30 wave capacitance gauge were installed on several occasions between August 2010 and February 2011 to examine the wave environment at Lake Pukaki. Due to installation difficulties that occurred in the field the WG-30 was not used for analysis. However, the installation is still described in Section 3.2 to illustrate the potential practicality of instrumental measurement of waves for wave recording in the South Islands alpine lakes.

3.2 Wave Recording Instruments

Direct wave measurements in lacustrine environments usually involve employing the use of a wave capacitance gauge and/or a pressure sensor. Initially a RBR XR-620 pressure sensor and a WG-30 wave capacitance gauge were set up in unison for wave recording at Lake Pukaki during August and September 2010. Wave recording commenced along the southern (Site 7 and Site 8, Figure 2.9) and the eastern shoreline (Site 5) when a strong northwest wind was forecasted for the Mackenzie Basin. The WG-30 was used to calibrate the effectiveness of using the XR-620 for measuring wave statistics. In previous studies (Pickrill, 1976; Allan, 1998; Dawe, 2006), the wave capacitance gauge has been successful at characterising the wave environment of the South Islands alpine lakes. The WG-30 has the capability of sampling at 10 Hz, which is sufficient to record high frequency waves in a restricted fetch lake environment. However, due to installation difficulties in the field, the measurements from the WG-30 were not used for wave analysis for this study. Table 3.1 summarises the wave recording events during the study period. On the two occasions the WG-30 did record, on the 13/08/10 and 17/09/10, calm conditions prevailed. Therefore, the data from the WG-30 was not useful. The XR-620 continued to record until February 2011 and, in total, was deployed on eight separate occasions. Nonetheless, only four of these measurement events have been included for analysis. They were on the 12/08/10, 18/12/10, 15/01/11 and 6/02/11. The other four occasions calm conditions persisted or errors were made when setting up the XR-620.

Table 3.1: Wave recording periods for the XR-620 pressure sensor and the WG-30 capacitance gauge at Lake Pukaki from July 2010 to February 2011.

Date	Location	XR-620 (6 Hz)	WG-30 (10 Hz)
28/07/10	Site 7	14:30 – 17:00 *Calm conditions*	-
12/08/10	Site 7	13:00 – 17:00 *Good data*	-
13/08/10	200 m west of Site 7	10:00 – 17:00 *Calm conditions*	11:30 – 15:30 *Calm conditions*
16/09/10 – 17/09/10	Site 7	13:00 (16/09) – 17:00 (17/09) *Recording failed, error occurred with internal clock*	11:00 – 16:00 (17/09) *Calm conditions*
18/11/10	Site 8	12:30 – 19:00 *Calm conditions*	-
18/12/10	Site 8	12:00 – 17:20 *Good data*	-
15/01/11	Site 8	12:20 – 20:20 *Good data*	-
6/02/11	Site 5	12:00 – 16:20 *Good data*	-

The RBR XR-620 (serial number: 12882) is a small, autonomous, submersible data logger. This product was developed in Canada by Richard Brancker Research Ltd (RBR). It has the capability for 24-bit profiling and sampling at 6 Hz. These multichannel loggers can be fitted with up to eleven different sensors. This particular model was fitted with a temperature, conductivity and pressure sensor. A Delrin® cylindrical plastic case encloses all the electrical circuitry. The logger is powered by four 3V CR123A batteries. For deployment the logger was moored to a concrete block weighing ~15 kg (Figure 3.1). An attached buoy was used to mark its position, as was a rope fixed to the shoreline. When deployed the unit sat at 0.3 m above the bed level in water depths of ~1 m.

The data recorded by the XR-620 that was pertinent for this study was supplied by the pressure sensor. The pressure sensor measured the total water surface variation and hence, the wave height and period. The temperature values recorded by the temperature sensor are also important because they are needed for air-water temperature differential calculations for wave hindcasting and forecasting that are presented in Chapter 4.

The WG-30 wave capacitance gauge, a popular device in lacustrine studies, was implemented to calibrate and determine the effectiveness of using the XR-620 pressure sensor for measuring wave heights at Lake Pukaki. The WG-30 was developed by the National Water Research Institute (NWRI) and manufactured by Richard Brancker Research Ltd (RBR) in Canada. These instruments are suited for use in low energy wave environments as they provide a very accurate, to 0.4%, record of the water surface fluctuations and ultimately, the wave form (Dawe, 2006). The WG-30 measures fluctuations in the water level by recording the variation in capacitance along a wire. The wire stands vertically adjacent to a telescopic aluminium pole that extends to 3.2 m in height. It is held taut by a spring loaded lever at the base and enters an electronic housing at the top enclosed by a plastic casing. When deployed at Lake Pukaki the WG-30 stood in ~1 m deep water, attached to a steel waratah that was driven into the lake bed. For further stabilisation, three guy wires situated on top of the instrument can be fixed to the lake floor. A DC power and data cable for the WG-30 unit connected to the electronic housing at the top. This cable lead back to a shore based Campbell Scientific CR23X data logger and a 12 V marine battery, which powered the device. The cable was held aloft by a steel waratah so it was not damaged within the swash zone. The entire set up for the WG-30 at Lake Pukaki is shown in Figure 3.2.

Figure 3.1: RBR XR-620 pressure sensor, moored to a concrete block ~15 kg. Orange buoy used to mark position when the instrument is submerged. Photo taken on 18/11/10.

Figure 3.2: WG-30 wave capacitance gauge set up. The WG-30 is attached to the steel waratah seen protruding above the lake surface. The Campbell Scientific CR23X data logger is housed in the blue chilly bin to the right of the photo. Orange buoy marks the location of the XR-620. Photo taken on 13/8/10.

It was noted by Allan (1998) and Dawe (2006), that there is no established methodology for measuring waves in alpine lakes, such as recommended sampling lengths and depths for device deployment. The literature shows that wave sampling lengths vary from ten minutes, over the course of an hour, through to a continuous time series. The duration of sampling is dependent upon the wave frequency spectrum being analysed. Restricted fetch environments, such as small lakes, are subject to high frequency waves.

The CR23x, utilised for the WG-30, and the XR-620 were programmed to record continuously over the recording period at a sampling rate of 10 Hz (0.1 s) and 6 Hz respectively. The data for the XR-620 would later be divided into twenty minute intervals for data analysis, which is discussed in Section 4.3. Continuous sampling for wave recording is uncommon due to its high power demand and the limited data storage of instruments. However, the duration of wave recording for a single deployment at Lake Pukaki never exceeded twenty four hours, so the battery life or data capacity was never jeopardised. The depth the wave recording instruments were deployed in was ~1 m. The wave induced pressures generated by high frequency waves are rapidly attenuated with depth (Allan, 1998; Dawe, 2006; Pedloksy, 2003). This concept applies to the XR-620 pressure transducer, which needed to be situated at a satisfactorily shallow depth to be able to detect the pressure signal arising from the shortest wave length of concern. According to Allan and Kirk (2000) a ~1 m depth provides good estimates for wave characteristics.

The introduction of pressure sensors for wave measurement in lakeshore environments is a relatively new concept. The reliability of employing pressure sensor devices for wave recording purposes in lakeshore environments is, however, subjective. Pressure sensor instruments utilised in previous lacustrine studies have not been able to display the entire wave form with great confidence. Sampling at low frequencies (~2 Hz), these instruments have required calibration with reliable wave recording instruments sampling at higher frequencies, such as the WG-30 capacitance staff. From this present study, it is considered that the XR-620, with the ability to sample at 6 Hz, is adequate for capturing the majority of the wave form for waves at Lake Pukaki and possibly for waves on other South Island alpine lakes. Figure 3.3 is a screenshot of the wave data collected on the 12/08/10, which is presented by the RBR Software (Version 6.08). The magnified wave form, shown in Figure 3.3b, shows the complete wave shape of an individual wave recorded by the XR-620.

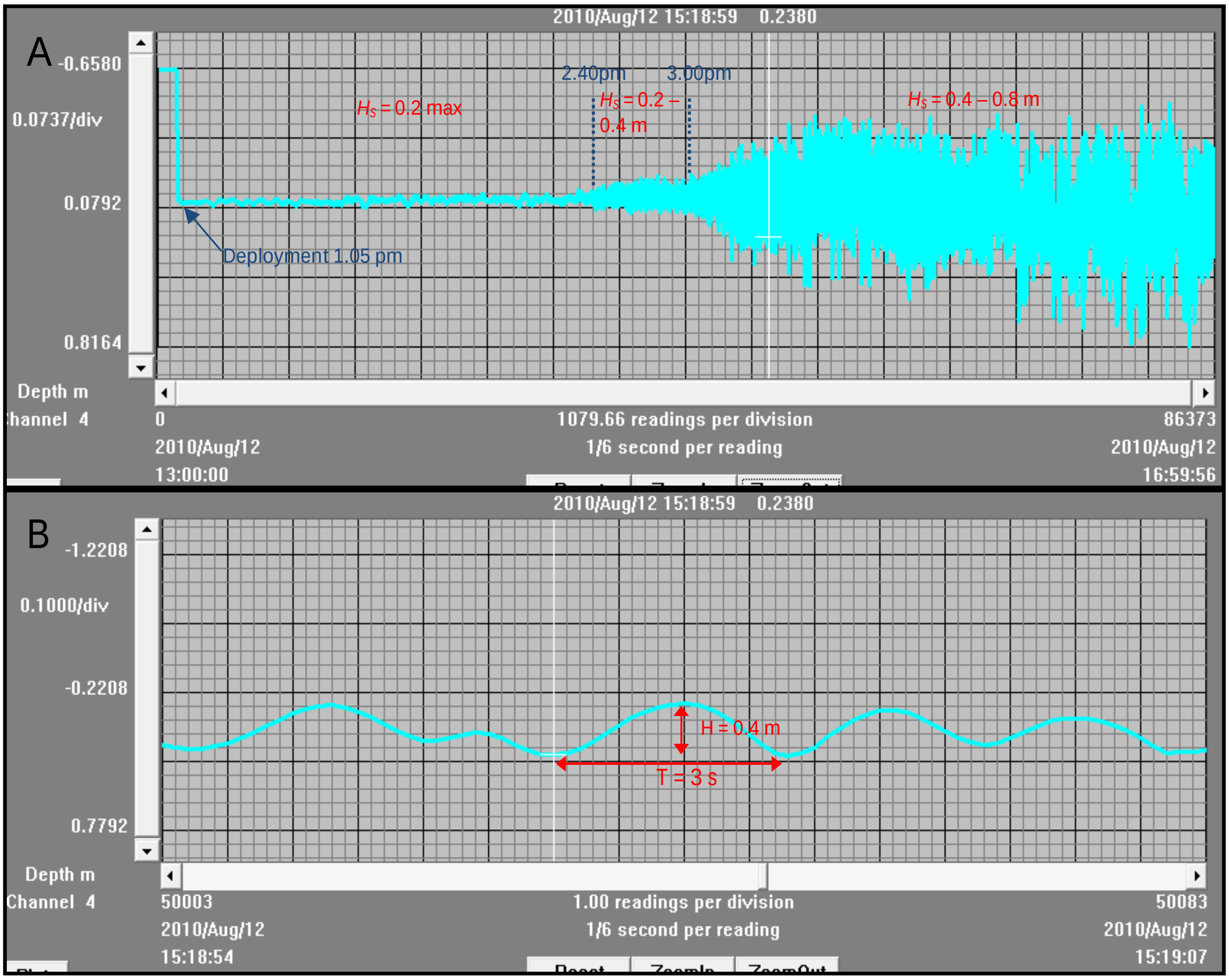


Figure 3.3: A) Wave spectrum recorded by the XR-620 on 12/08/10 from 13:00 to 17:00 at Site 7. The south station received increased wind speeds from the north at 14:40, which explains the amplification in wave height at this time. B) Magnified wave form at 15:18:59. H_S is the significant wave height, H is wave height and T is wave period.

3.3 Raw Data Analysis

Upon completion of sampling the data collected by the XR-620 is downloaded to a laptop or desktop computer and stored as a .DAT file. Before statistical analysis the surface wave data needs to be calculated from the pressure readings. Unfortunately, the RBR software (Version 6.08) does not have the ability to compute this so a routine within MATLAB was developed. The steps required to obtain the surface wave data are explained below. For ease of analysis and compatibility for the MATLAB routine, the wave data was divided into twenty minute intervals. The layout for an example .DAT file used for the MATLAB routine is shown in Appendix 1.

Firstly, the pressure values are converted to depth values. The recorded pressure reading (P_R) has to be adjusted to the atmospheric pressure (P_A). The atmospheric pressure used was taken from the southern climate station during the time the submersible data logger was recording. Unfortunately, the south station only recorded the atmospheric pressure during one wave recorded period, on the 12/8/10. A value of 9.418968 dbar has subsequently been applied for all four wave recording occasions, which is the average pressure value recorded by the south station.

$$P_W = P_R - P_A \qquad \text{Equation 3.1}$$

Where P_W is the water pressure (dbar). A simple calculation is then required to obtain the depth values.

$$d = \frac{P_W}{0.980665\,\rho_w} \qquad \text{Equation 3.2}$$

Where d is the depth (m) and ρ_w is the density of the water (kg/m^3). A different value of ρ_w is applied for each of the four separate wave recording events, as the water density is dependant on the water temperature. A more complicated method exists for calculating the depth from the pressure data but can only be applied when sampling in salt water.

The MATLAB routine utilises the Fast Fourier Transform (FFT) method of spectral analysis. A standard FFT algorithm is performed to determine the Fourier coefficients of the depth time series. The next key step is to remove the attenuation of surface waves that occurs with increasing depth (Dr. F. Johnson, RBR Ltd 2010, <u>pers. comm.</u>). The attenuation function is computed over the spectral range for the devices deployment depth and then this function is convolved with the original data. Following this an inverse FFT is applied to give the surface wave. The reader is directed to Pedlosky (2003) for a detailed explanation of the steps outlined above.

3.4 Results of Wave Measurement at Lake Pukaki

The surface wave data is statistically analysed to resolve common deepwater wave parameters. These parameters are presented in Appendix 2. In all, the XR-620 recorded sixty three twenty minute sample periods, with only fifty seven being chosen for analysis as they were representative of storm conditions. The remaining six records were of waves attaining a significant wave height (H_s) of below 0.08 m and were excluded from the analysis due to the small amount of geomorphic work resulting from these waves. The significant wave height (H_s) is the average height of the highest one third of waves. While more wave measurements at different sites would have been advantageous it was impractical to have achieved this within the given timeframe. As this study is concerned with the maximum potential wave energy arriving at the shoreline the majority of the wave recording has been completed along the southern shoreline at Site 7 and Site 8, exposed sites on the lake. The southern shoreline is exposed to the longest fetches and to the strong gusty north/north-westerly winds, producing maximum wave heights at this location. Wave recording was also carried out on the eastern shoreline on one occasion (6/02/11). This data will assist in explaining the variability in wave climate geographically along the length of Lake Pukaki, especially when correlating with modelled wave statistics later discussed in Section 4.5.

3.4.1 Wave Height

The summary statistics for the deepwater wave height data are shown in Table 3.2. The mean H_s was 0.53 m, with a minimum of 0.09 m and a maximum of 1.07 m. These values compare well with visual observations carried out at Lake Pukaki by Allan (1991), where he observed H_s ranging between 0.2 m and 1.2 m, with a mean of 0.57 m. H_{max} is the maximum wave height for a specified period of time, in this case every twenty minute interval. The largest recorded wave height of 1.84 m occurred along the southern shore at Site 8 on the 15/01/11. The mean H_{max} was 0.9 m with a minimum of 0.22 m. Allan's (1991) observations also compare favourably with measured H_{max} values for this study. He observed H_{max} reaching 2.5 m with a minimum of 0.3 m and a mean of 0.96 m. Although, his observed maximum H_{max} of 2.5 m exceeded the maximum measured for this study by as much as 0.66

m. It is difficult to clarify the variance between the maximum H_{max} values for this study and Allan's (1991) due to the fact that they were both measured during an average wind speed of 10 ms^{-1} at locations along the southern shoreline only 1 km from each other.

The wave height statistics are slightly positively skewed and platykurtic, which indicates that the wave heights are well distributed across the size range but exhibit a small tail of large wave heights. This degree of skewness is typical in wave data sets, as reported by Dawe (2006) at Lake Coleridge, being referred to as a Rayleigh distribution.

Table 3.2: Summary statistics of the wave height parameters measured by the XR-620, 6 Hz, n = 57. H_s is the significant wave height, H_{10} is the average height of the highest one tenth of waves, H_{max} is the maximum wave height and H_{rms} is the root mean square wave height. The figures provided by Pickrill (1976), Allan (1998) and Dawe (2006) are the mean values whereas the H_{max} are the maximum values.

	H_s	H_{10}	H_{max}	H_{rms}
Mean	0.53	0.66	0.90	0.38
Minimum	0.09	0.12	0.22	0.06
Maximum	1.07	1.28	1.84	0.78
Skewness	0.15	0.13	0.21	0.14
Kurtosis	-1.42	-1.50	-1.46	-1.38
Variance	0.07	0.12	0.20	0.04
Std. Deviation	0.27	0.34	0.45	0.20
Pickrill (1976) n = 47, 3.3 Hz	0.16	-	-	0.12
Allan (1998) n = 53, 2 Hz	0.28	0.34	1.05	0.16
Dawe (2006) n = 493, 10 Hz	0.17	0.22	0.84	0.20

A few wave parameters obtained by Pickrill (1976), Allan (1998) and Dawe (2006) have also been included in Table 3.2. The H_s values attained within this study are considerably greater than what was found on other South Island alpine lakes. Both Pickrill's (1976) and Dawe's (2006) H_s values are very similar, whereas Allan (1998) acquired a much larger H_s in comparison. Allan's (1998) wave sampling was confined to storm conditions, as with this study, which may explain the higher obtained H_s values. Lake Pukaki could also be experiencing a higher wave energy environment, which is further reinforced by the largest recorded wave of 1.84 m found from this study. Lake Pukaki has a longer effective fetch than Lakes Manapouri, Dunstan and Coleridge, which is probably the main reason for the larger wave heights. The wind speeds at Lake Pukaki could also be stronger than the other mentioned sites, which is further discussed in Section 4.3.

Significant wave height (H_s) has some well known relationships with other common wave height statistics (Demirbilek and Vincent, 2002):

$$H_s = 1.416\, H_{rms}$$

$$H_{10} = 1.27\, H_s$$

$$H_{max} = 1.86\, H_s$$

Where H_{rms} is the root mean square wave height and H_{10} is the average height of the highest one tenth of waves. Specified for the coastal environment, the H_s relationships identified above show a good correlation when applied to the wave data collected from this study at Lake Pukaki. Linear regression analysis was employed to compare predicted wave variables (H_{rms}, H_{10} and H_{max}) with measured values and an R^2 of 0.98 – 1 was obtained. It is important to note that the H_{max} formula is intended for one thousand wave cycles in one record. Nevertheless, this relationship still displays a thorough correlation, with each twenty minute interval containing between 316 and 729 waves. Having recognised that these wave relationships abide accordingly to the Lake Pukaki setting, wave variables can be determined to a reasonable degree of accuracy with the knowledge of only one variable (i.e. H_s). However, this may only apply to storm conditions at Lake Pukaki.

3.4.2 Wave Period

Table 3.3 summarises all of the deepwater wave period statistics acquired by the XR-620 at Lake Pukaki. Significant wave period (T_s) is the average period of the one-third highest waves of a given wave group. T_s ranged between 1.57 and 4.51 s, with a mean of 3.26 s. The zero crossing period (T_z) is the average period of the zero-up-crossing waves of a given wave group. The mean T_z was 2.83 s, with a range of 1.64 to 3.8 s. As shown with the wave height, Allan's (1991) visual observations for wave period at Lake Pukaki correlate well, which ranged from 1 to 4 s, with a mean of 2.88 s. All the wave period data is negatively skewed and platykurtic, indicating they are well distributed across the spectrum but display a large tail of shorter wave periods. When compared to measurements by Pickrill (1976), Allan (1998) and Dawe (2006), Lake Pukaki experiences larger wave periods than Manapouri, Dunstan and Coleridge respectively. This is reflective of the longer fetch lengths at Lake Pukaki and stronger wind speeds when compared to Lakes Manapouri, Te Anau, Dunstan and Coleridge.

Table 3.3: Summary statistics for the wave period parameters measured by the XR-620, n = 57. T_s is the significant wave period, T_z is the zero crossing period, T_c is the wave crest period and ε is the spectral band width. Mean values acquired by Pickrill (1976), Allan (1998) and Dawe (2006) are also included for comparison.

	T_s	T_z	T_c	$\boldsymbol{\varepsilon}$
Mean	3.26	2.83	2.09	0.35
Minimum	1.57	1.64	1.38	0.03
Maximum	4.51	3.80	2.75	0.79
Skewness	-0.43	-0.22	-0.52	0.4
Kurtosis	-0.61	-1.02	1.43	-0.22
Variance	0.53	0.30	0.06	0.03
Std. Deviation	0.73	0.55	0.25	0.18
Pickrill (1976) n = 47, 3.3 Hz	-	1.91	1.69	0.43
Allan (1998) n = 163, 2 Hz	2.54	2.18	2.00	0.34
Dawe (2006) n = 493, 10 Hz	2.03	1.43	0.75	0.84

The spectral band width (ε) (dimensionless) is the measure of the range of frequencies present in a wave record relative to the average wave frequency (CERC, 1984a). In other words, it is used to determine the narrowness of the wave spectrum (Demirbilek and Vincent, 2002), and is derived by:

$$\varepsilon = \sqrt{1 - \left(\frac{T_z}{T_c}\right)^2} \qquad \text{Equation 3.3}$$

Where T_c is the wave crest period. T_c is determined by the time (in seconds) of the sampling interval, divided by the number of wave crests in that specified period. ε values closer to zero indicate a narrow banded spectrum, associated with steady wind conditions. In contrast, values nearer to one signify a mixed wave state that has a wide range of wave periods, commonly associated with storm conditions (Dawe, 2006). Figure 3.4 demonstrates that increases in wave height are proportional to increases in the spectral band width. This implies that larger waves at Lake Pukaki are associated with storm events. Overall, Lake Pukaki displays highly variable conditions, as ε ranges between 0.03 and 0.79. Although, as with Pickrill (1976) and Allan (1998), Lake Pukaki is more affiliated with swell conditions, referring to its mean ε of 0.35. Dawe (2006) found that Lake Coleridge is representative of storm conditions, regarding a mean ε of 0.84. He obtained a much higher ε value because the wave gauge he used, sampling at 10 Hz, had the ability to capture the more frequent wave crests.

These findings have identified a flaw with using a pressure sensor for wave measurement in a lacustrine environment as they are unable to detect the higher frequency wave crests, resulting in a lower T_c, thus a smaller ε value.

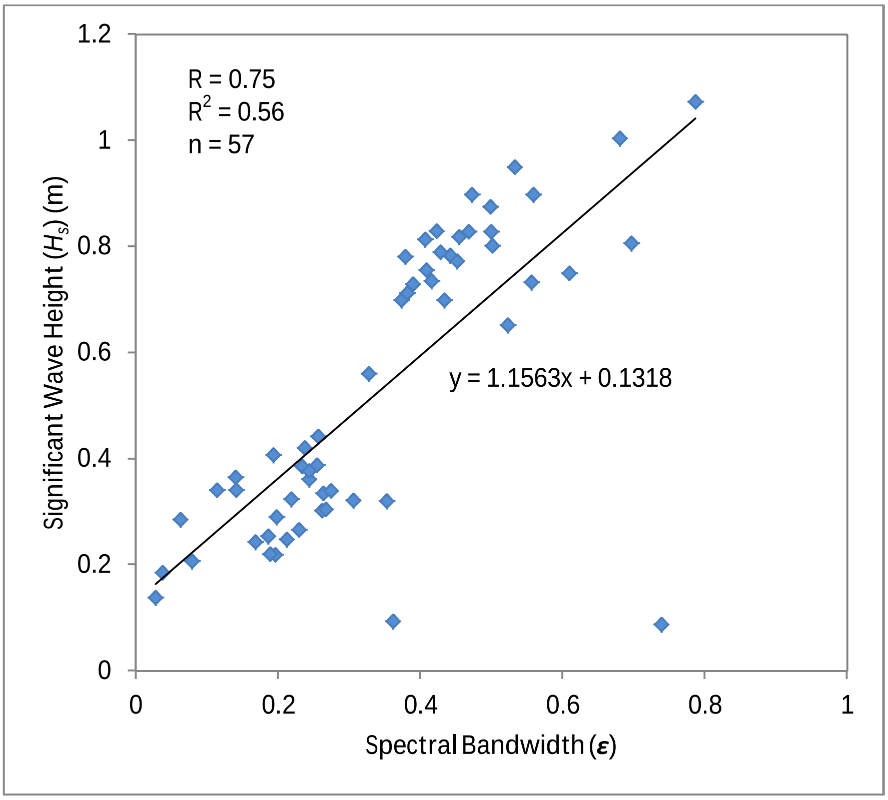


Figure 3.4: Comparison of the spectral band width (ε) and the significant wave height (H_s).

3.4.3 Wave Length and Steepness

The deepwater wave length can be determined from the following equation using linear wave theory:

$$L_o = \frac{gT^2}{2\pi} \qquad \text{Equation 3.4}$$

Where L_o is the deepwater wave length (m). The zero crossing period (T_z) is used to calculate L_o. Mean wave lengths were 13 m, with a range of 4.22 to 22.49 m. These values are considerably greater than what has been recorded on other South Island alpine lakes: Pickrill (1976), Allan (1998) and Dawe (2006) approximated mean figures of L_o between 3.26 and 7.17 m. However, they are comparable to visual observations by Allan (1991) at Lake Pukaki.

Wave steepness is measured by the ratio of wave height to wave length (H/L). Waves displaying a low wave height to wave length ratio are commonly referred to as swell waves and are depositional, leading to shoreline accretion. In contrast, steep waves tend to erode the shoreline resulting in beach regression. Erosive steep waves are also commonly associated with storm events. Lake waves on the South Island alpine lakes are considered to be steep and erosive, in particular at Lake Pukaki (Kirk, 1988; Allan, 1991). Compared to the coastal environment, lake waves constitute more of a vertical energy component due to the existence of a steeper foreshore and steeper waves.

Table 3.4 presents a comparison of the wave steepness values acquired by Pickrill (1976), Allan (1998), Dawe (2006) and the present study. To achieve a sound degree of validity the present and the previous studies shown in Table 3.4 use the significant wave height (H_s) and the wave length determined by the zero crossing period (T_z) to ascertain the wave steepness. Implementing T_s for determining L_o produces a lower value for wave steepness. With a mean wave steepness of 0.039 and a range of 0.01 to 0.053, these values compare favourably with the other authors wave steepness values specified in Table 3.4. The slight variance in steepness values is dependent on the device utilised to measure waves. Dawe

(2006) attributes the higher mean value obtained from Lake Coleridge solely due to the faster sampling rate (10 Hz) of the wave gauge detecting more high frequency waves. Kirk (1970) stated that for short period waves the limiting steepness is around 0.076. This upper limit seems to agree with every author except for the present study. The maximum steepness of 0.053 could be owing to longer periods measured by the XR-620 at Lake Pukaki or due to a deficient sample size. There is however a possibility that the wave steepness values presented here are representative of Lake Pukaki. Different site characteristics of the other South Island lakes, including fetch lengths and nearshore bathymetry, may explain the variance of wave steepness values. The steepest waves at Lake Pukaki were measured at Site 8, which is exposed to the longest fetch length situated on the southern shoreline.

Table 3.4: Comparison of wave steepness values for the South Island alpine lakes. WG is the wave capacitance gauge, S4 is the InterOcean S4ADW directional wave and current recorder and XR is the XR-620 pressure sensor.

Wave Steepness (H/L)	Minimum	Max	Mean
Pickrill (1976) (WG, 3.3 Hz)	0.012	0.077	0.031
Allan (1998) (WG & S4, 2 Hz)	0.011	0.077	0.036
Dawe (2006) (WG, 10 Hz)	0.010	0.074	0.051
This study (XR, 6 Hz)	0.010	0.053	0.039

3.4.4 Breaker Wave Types

Wave breaking is dependant upon the wave steepness and beach slope. There are four basic types of breaking water waves: spilling, plunging, collapsing and surging. The associated beach slope for the four mentioned breaker types steepens respectively. Collapsing waves are considered as the transitional stage of plunging to surging waves. The breaker type generally defines the type of work done on a beach; whether it is erosion or accretion.

Many breaker wave type formulas exist, however Dawe (2006) determined that the Iribarren parameters (ξ), also known as the surf similarity, provided the best indication of breaker types at Lake Coleridge, in particular the deep water wave Iribarren (ξ_o). The same formula has been applied for this study to calculate breaker types at Lake Pukaki, which is shown below. It is a dimensionless ratio between the beach slope (*β*, in degrees) and the wave steepness.

$$\xi_o = \frac{\tan\beta}{\left(H_o/L_o\right)^{0.5}} \qquad \text{Equation 3.5}$$

Utilising the wave data obtained from the XR-620 the breaker wave types for Lake Pukaki were ascertained and are presented in Figure 3.5. As the wave form undergoes minor alteration in the steep nearshore zone the significant wave height (H_s) was used for the deepwater wave height (H_o).

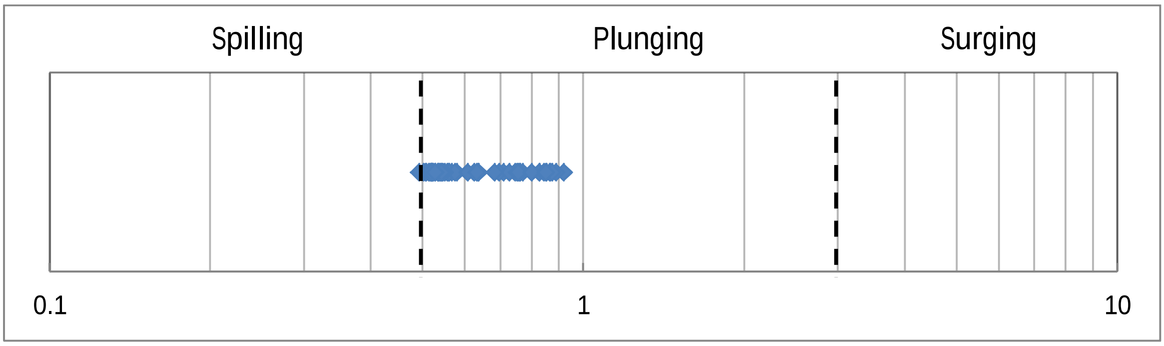


Figure 3.5: Deep water wave Iribarren (ξ_o) using the XR-620 wave data.

It is evident from Figure 3.5, which matches field observations, that plunging waves are the most common breaker type at Lake Pukaki. Plunging waves are also very common on other South Island alpine lakes including Lake Dunstan (Allan, 1998), Lakes Manapouri and Te Anau (Pickrill, 1976) and Lake Coleridge (Dawe, 2006). Waves of this form on steep alpine beaches are able to break very close to the shore and release most of their energy within a narrow zone of impact. Allan (1991) noted that plunging waves tended to have smaller swash cycles on Lake Pukaki, and were highly erosive to the beach morphology. The front face of a plunging wave steepens, becomes vertical and eventually the crest begins to curl over and drops on the trough of the wave (Figure 3.6). Typically they occur as a single line of plunging breakers that is consistent with open-coast mixed sand and gravel beaches.

Figure 3.6: Plunging waves west of Site 9. Photo taken on 6/02/11.

3.5 Conclusions

This chapter has provided insight for the wave processes active at Lake Pukaki. It has presented the results and provided a discussion of the deep water wave measurements. The only other documented study carried out at Lake Pukaki regarding the wave environment was by Allan (1991). The majority of his analysis was based on visual observations and hindcasted wave data. This thesis marks the first occurrence where waves were directly measured via instrumentation at Lake Pukaki. The XR-620's sampling rate of 6 Hz seemed to capture the wave form sufficiently.

The mean significant wave height (H_s) identified for this study was 0.53 m, while the maximum recorded wave height was 1.84 m. When compared with studies conducted by Pickrill (1976), Allan (1998) and Dawe (2006) on other South Island alpine lakes, Lake Pukaki's wave spectrum is capable of achieving much greater wave heights and large wave periods (1.8 – 3.8 s). The associated wave lengths (L_o) are also very large in comparison, averaging 13 m, which have developed primarily due to a longer effective fetch length at Lake Pukaki. The values obtained for the spectral bandwidth parameter (ε) suggest that Lake Pukaki is associated with swell conditions, which is the opposite of what was initially expected and observed in the field. This has highlighted a defect with using a pressure sensor, such as the XR-620, for the recording of high frequency waves in a restricted fetch environment.

Alpine lakes, including Lake Pukaki, are characteristic of steep plunging waves. Being highly erosional in nature, these waves are able to break very close to the shore owing to the steep gradient of the foreshore. Alpine lake waves exhibit more of a vertical energy component, as opposed to open-coast waves, as wave energy is concentrated in a narrow zone just in front of the trough as the wave breaks.

The understanding of the deepwater wave characteristics at Lake Pukaki is essential for validating the reliability of modelling wave data, which is evaluated in Chapter 4. Knowledge of nearshore waves is also necessary for examining and analysing the performance of shore protection structures, as will be discussed further in Chapter 6.

Chapter 4

WINDS AND WAVE MODELLING

4.1 Introduction

This chapter comprises four parts. First, it presents the concept and models utilised for wave hindcasting. Secondly, it describes the wind regime around Lake Pukaki over the field observation period from July 2010 to February 2011 and compares and contextualises this in relation to nearby weather stations. Thirdly, it discusses the modelled wave environment via the implementation of adjusted wind data for wave hindcasting. Finally, the reliability of wave prediction at Lake Pukaki is determined via correlation with measured wave data.

There has been a paucity of direct wave recording on New Zealand lakes, with the exclusion of studies by Pickrill (1976), Allan (1998) and Dawe (2006). Indirect methods such as wave hindcasting/forecasting from wind observations have been the method of reliance. Hindcasting refers to wave statistics derived from past meteorological conditions while forecasting relates to the wave conditions to be expected from given or predicted wind conditions (Kirk, 1988). The same procedures for generating modelled waves are applied in both cases, the only difference being the source of wind data used.

This study has implemented wave prediction techniques utilising wind data. Due to the time constraints of the study period, wave characteristics could not be directly measured along the entire shoreline of Lake Pukaki, but could be modelled via wave hindcasting. It is important though that the theoretical assumptions presented by the wave hindcast models are correlated with measured wave statistics to determine the model's ability to predict waves in an alpine lake environment, such as Lake Pukaki. Wave data measured from the RBR XR-620 pressure sensor is used for correlation with the wind-wave estimates and is evaluated in Section 4.5. Measuring berm heights on lake beaches has also proven to be an effective way of cross checking predictions based on wind data (Kirk, 1988, Kirk *et al*. 2000). This method is based on the knowledge that berms are formed by deposition of sediment to the maximum elevation reached by the runup of the largest waves. However, this latter method was not carried out for this study.

Taking into account the significant relationship between wind and wave generation on lakes, two weather stations were set up in close proximity to Lake Pukaki in order to analyse the wind system. Previous lacustrine studies have relied on weather records for wave hindcasting purposes, from weather stations that are not located near or along the lakeshore. There is a high level of uncertainty involved due to the fact that the weather data is not representative of the lakeshore environment. This is particularly true for alpine lakes whereby the winds are subject to topographic forcing.

The ability to hindcast waves within an alpine lake environment requires an assessment of the local topography and hence an understanding of the complexity of the wind system (Dawe, 2006). Examination of wind observations within a one year period is likely to result in a misinterpretation of the wind regime as not all wind conditions may be represented in the relatively short measurement period. For this reason, it is important to interpret weather records, from nearby stations, for at least a decade to identify any localised wind phenomena and to include as much long-term variability of the wind regime as possible. Findings from this study are also examined in relation to wind records and wave statistics from previous studies on Lake Pukaki by Bunting (1977), Kirk (1988) and Allan (1991).

In agreement with Allan's (1991) findings, the author emphasises the importance of topographically channelled north/north-westerly winds for wave development at Lake Pukaki for three reasons. Firstly, winds from this direction blow for long periods; secondly, north/north-westerly winds blow across the longest fetch of the lake; and thirdly, strong to gale force winds originate from the north/north-westerly direction. The short-period high-magnitude storm events, eventuating from a strong north/northwest wind flow, tend to cause the most significant erosion along the shoreline (Kirk, 1988; Allan, 1991).

4.2 Review of Wave Hindcasting Models

LAKEWAVE is a wind-wave hindcast model developed by Hicks (2000), for restricted fetch, enclosed waterways such as lakes and harbours. It is based on the deepwater wave hindcasting, narrow fetch model NARFET by Smith (1991). NARFET was formulated on the performance of four wave hindcasting models in use at the time. Directly measured wave statistics were obtained from field data on the Great Lakes in North America and were correlated with the four hindcast models to ascertain the suitability for predicting wave characteristics. These models included the effective fetch and simple fetch methods described in CERC (1984a), the Donolan (1980) model, which used a reduced wind driving component for off wind directions, and the Walsh *et al.* (1989) model. In New Zealand the effective fetch method was used by Pickrill (1976) and MacBeth (1988), whereas the simple fetch model was implemented by Kirk (1988) and Allan (1991) at Lake Pukaki. Smith's (1991) findings revealed that the effective fetch method performed poorly, underestimating wave statistics and was excluded from further analysis. The simple fetch method provided reasonable agreement between the measured and hindcast waves, however the Donolan model provided the closest estimate of wave period and the Walsh *et al.* model predicted wave height the best. Based on her analysis, Smith (1991) reserved the Donolan idea of wave development in off-wind directions and developed new equations for estimating wave height and wave period. The equations used in the NARFET model are as follows:

$$H_s = 0.0015\, g^{-0.5}\, F^{0.5}\, (U \cos\phi) \qquad \text{Equation 4.1}$$

$$f_P = 2.6\, g^{0.72}\, F^{-0.28}\, (U \cos\phi)^{-0.44} \qquad \text{Equation 4.2}$$

Where f_p is the peak frequency (Hz), g is the acceleration due to gravity (9.81 ms^{-2}) and ø is the angle between the wind and off wind direction (º).

The output values for the LAKEWAVE model are the deepwater significant wave height (H_{so}), wave period (T) and direction. The wave approach angle is incorporated in predictions for longshore sediment transport rates, described in Section 5.5. A comparison of hincasted H_{so} and T versus measured wave properties are presented in Section 4.6.

Allan (1998) utilised the NARFET model at Lake Dunstan and concluded that the model performed adequately for hindcasting wave heights for two of his four study sites. The other two sites NARFET tended to over predict wave height by as much as 53%. Overall there was a poor correlation with wave period. Nevertheless, LAKEWAVE, which utilises the NARFET model, can be considered currently the best available option for wave hindcasting in small lake environments.

The assumptions for the LAKEWAVE model are that waves are locally generated and fetch limited; the enclosed water body is deep, so that the depth is greater than half the wavelength of the peak energy frequency, except in the nearshore; the wind field is uniform and is represented by records from a single station; wind conditions are steady, equalling the average condition between records; and the waves are not diffracted or refracted, except for simple refraction of shoaling waves approaching the shore defined by smooth, parallel contours.

Kirk (1988, p 92) stated that:

> *"Whatever method is employed it is important to realise that the complexity of a wind-aroused lake surface is much greater than what is implied by theory and that this complexity is commonly represented by just a few parameters."*

This statement further reinforces the importance of comparing hindcasted waves with measured wave characteristics. The applicability of using wind data for determining wave characteristics along the entire stretch of shoreline of Lake Pukaki is dependent on the agreement shown between the hindcasted and measured wave statistics.

Program Input

Two data files are required for the LAKEWAVE program to operate. A shoreline data file (*.SHO) is needed, which presents the two dimensional coordinates of the shoreline and any islands within the water body. The easting and northing data are in metres and the shoreline points advance clockwise around the shore. The distance between successive points should be approximately 200 m. The second necessary component is the wind data, which can either be keyed in one event at a time or input as a time series data (*.DAT) file. The time series file includes the average wind speed and direction, where the interval between records is assumed to be uniform, in this case hourly. Wind data are adjusted (U_A) for the elevation at which the measurements were taken; whether the measurements were taken over land or over water; and a stability correction factor based on the air-water temperature differential, ΔT_{AW}. Derivation of U_A (adjusted wind speed) is discussed in Section 4.4 and described fully in CERC (1984a, 3-26).

Essentially, there are five factors that need to be considered when using wind data for wave hindcasting. The wind speed, wind direction, wind frequency (duration), fetch length and the water depth. James *et al.* (2002) discredits the use of water depth when applying this method to alpine lakes as they are relatively deep and consist of narrow nearshore shelves. For LAKEWAVE, all the wind variables are given within the wind time series data and radial fetch lengths to the point of interest are calculated from the digital shoreline files. The elevation of the lake level in relation to the coordinates outlining the perimeter of the lake, from the shoreline file, is unknown. For analysis, it is assumed that the shoreline file provided for Lake Pukaki represents the maximum lake level of 532 msl (metres above mean sea level). The LAKEWAVE model outputs values of deepwater significant wave height (H_{so}), wave period (T) and wave direction, which are presented and reviewed in Section 4.5. Other variables such as breaker height (H_b), wave length (L), longshore transport rates and wave runup are calculated based on the values of H_{so}, T and wave direction. Due to their dependency on H_{so}, T and wave direction, longshore transport and wave runup values are discussed in Chapter 5 following the correlation of hindcasted and directly measured waves.

4.3 Wind Regime of Lake Pukaki

4.3.1 Overview

Foehn northwesterly winds, which are locally referred to as the nor'wester, are the most familiar form of airflow occurring within the Mackenzie Basin. They develop due to a regular procession of anticyclones and low pressure troughs moving eastwards from the Tasman Sea, in the west (Bunting, 1977). The north-westerly winds, preceding the advance of the low pressure troughs, cross over onto the leeward side of the Southern Alps and descend into the deeply incised valley networks. The channelling of the foehn airstream down the Tasman, Hooker and Jollie Valleys results in the jetting of low level airflow, which enters Lake Pukaki at its northern end. The same phenomenon takes place at Lake Tekapo originating within the Godley and Macaulay River Valleys (McGowan *et al*. 1996). Kerr (2009) described the hydrology of the Lake Pukaki catchment as predominantly controlled by the magnitude of the north-westerly winds. The foehn effect is illustrated by the steep rainfall gradient in the upper regions of the catchment, near Mt Cook, as discussed previously in Section 1.3.4.

4.3.2 Wind Data Analysis: July 2010 – February 2011

Weather Station Installment

Two automatic HOBO climate stations were installed on the southern and eastern shores, their locations are shown in Figure 4.3. The east station (Figure 4.1) was located on an exposed bluff adjacent to the lake 20.5 km along Hayman Road, north of the SH 8 intersection. A far greater amount of vehicle traffic is encountered along SH8 on the southern shoreline so a decision was made that the south station (Figure 4.2) be situated at a higher elevation south of the lake because of the lower likelihood of it being interfered with by the public. The east and south stations location, recording periods and data collection types are summarised in Table 4.1.

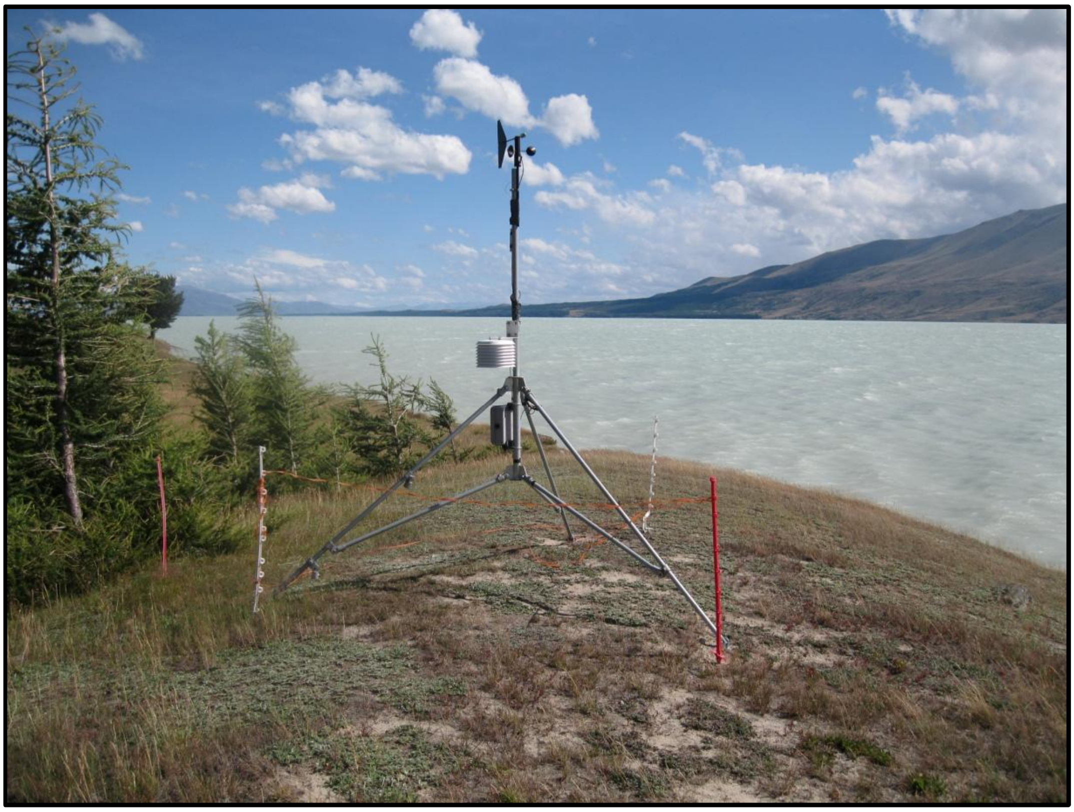


Figure 4.1: East weather station surrounded by a temporary electric fence, looking southwest. Photo taken on the 6/2/11 during a strong northerly wind. These winds have developed a succession of wave crests heading south (towards the left of this photo). Evidence of recurring gusty northerly winds is shown by the growth characteristics of the trees branches in the background.

Figure 4.2: South weather station, looking south. Photo taken on the 29/10/10. The steel wire mesh, to the left of the picture, was used as a fence for the weather station. It is important to note the hillside in the background, encompassing the southern side of the weather station, which may have had an effect on the wind pattern.

Table 4.1: HOBO weather station summary at Lake Pukaki.

ID	Coordinates NZTM	Recording Period	Data Types Collected
East Station	E 1375132 N 5122393	28/07/10 – 6/02/11	Wind speed, wind direction, air temperature, humidity.
South Station	E 1373012 N 5104196	27/07/10 – 29/10/10	Wind speed, wind direction, air temperature, humidity, atmospheric pressure.

Each climate station comprised of an adjustable steel pole tripod. A three cup anemometer and a wind vane were attached to the top of the central pole, which stood 2.5 m above the ground surface. A humidity and temperature sensor, a pressure sensor and a data logger unit were also fitted. A pressure sensor was only installed on the southern station. Data was sampled for sixty seconds and averaged at ten minute intervals. Three second wind gusts were also recorded. A 1 m high wire mesh fence was erected around the southern station for protection from rabbits and other pests. A temporary electric fence was installed for the eastern weather station by the farm owner due to the presence of nearby grazing stock. Data was downloaded from the data logger usually on a monthly basis. This ensured that, if for some reason the weather station stopped recording while unattended, the entire data set was not lost. To backup the weather station measurements, average readings of wind speed during a one minute period were taken, on several occasions when in the field, using a hand-held Kestrel 3000 digital anemometer.

Only one problem arose in relation to the climate stations. During a visit on the 16/9/10 a screw had to be replaced on the eastern station after the wind vane had become a bit loose. The eastern and southern station data sets were compared and it was concluded that the eastern stations north point had shifted approximately 10° to the east. Knowing this the data could be corrected. Lining up the north point of the climate stations using a handheld compass proved to be difficult. The metal tripod affected the magnetism of the needle on the compass. For this reason true north was difficult to determine. The wind direction was adjusted in the digital dataset if the data seemed incorrect.

Wind Results

The highest average wind speed over a ten minute interval was 31.4 ms^{-1} on the 24/9/10 between 10:50 and 11:00 recorded by the east weather station. This particular time also produced the greatest wind gust of 39 ms^{-1}. These wind speeds correlate well with Allan's (1991) findings, in which he recorded wind speeds of up to 40 ms^{-1} at Lake Pukaki. Whereas, the south station maximum recorded wind speed was only 16 ms^{-1} with a highest wind gust of 25.79 ms^{-1}. No winds stronger than 20 ms^{-1} (45 mph) were recorded in the raw wind data by Bunting (1977) from March 1970 to July 1976 at Lake Pukaki. However, the representativeness of Bunting's (1977) wind data is questionable as only 65% of the total wind records were available for his analysis.

For analysis purposes the raw wind data for 2010 and early 2011 was averaged over hourly periods. Appendix 3 includes tables summarising the hourly distribution of wind data by month for both the weather stations. A seasonal trend is evident with the stronger wind events occurring around the spring equinox in September and the summer solstice in December. The seasonality on an annual basis is inconclusive as only up to 193 days of recordings are presented for analysis. However, research carried out by Bunting (1977), Kirk (1988) and Allan (1991) at Lake Pukaki all suggest that spring and summer are the windiest periods, with winter being the calmest.

Kirk (1988) proposed a minimum threshold for which winds were regarded as significant for lakeshore erosion. This lower limit was for wind speeds below Force 4 (5.7 – 8.8 ms^{-1} or 11 – 16 knots), referring to the beaufort wind scale. From Kirk's (1988) analysis only 11.66% of winds were considered as important for lakeshore change. Allan (1991) found that winds blew equal to or in excess of Force 4 30.39% of the time at the north end of the lake near Glentanner and 24.31% at the southeast corner close to the Tekapo B Power Station. In regards to the same wind speed bracket for the 2010 dataset, the east stations winds blew equal to or in excess of Force 4 for 21.29% of the time and the south station for only 9.67% of the time. These values compare well to Allan's (1991) findings, especially the Tekapo B site and the east station. This is likely due to the close proximity of the two stations.

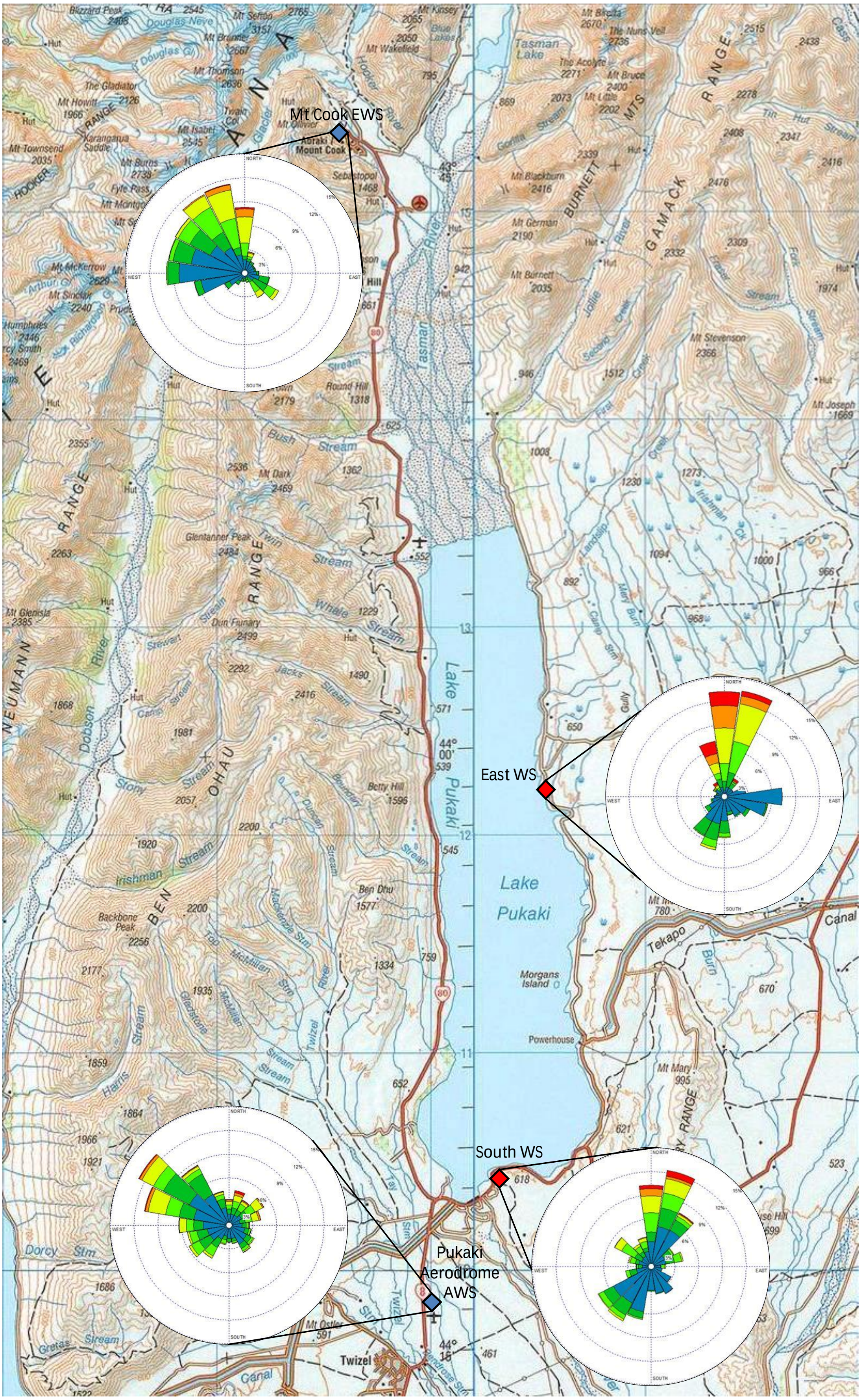
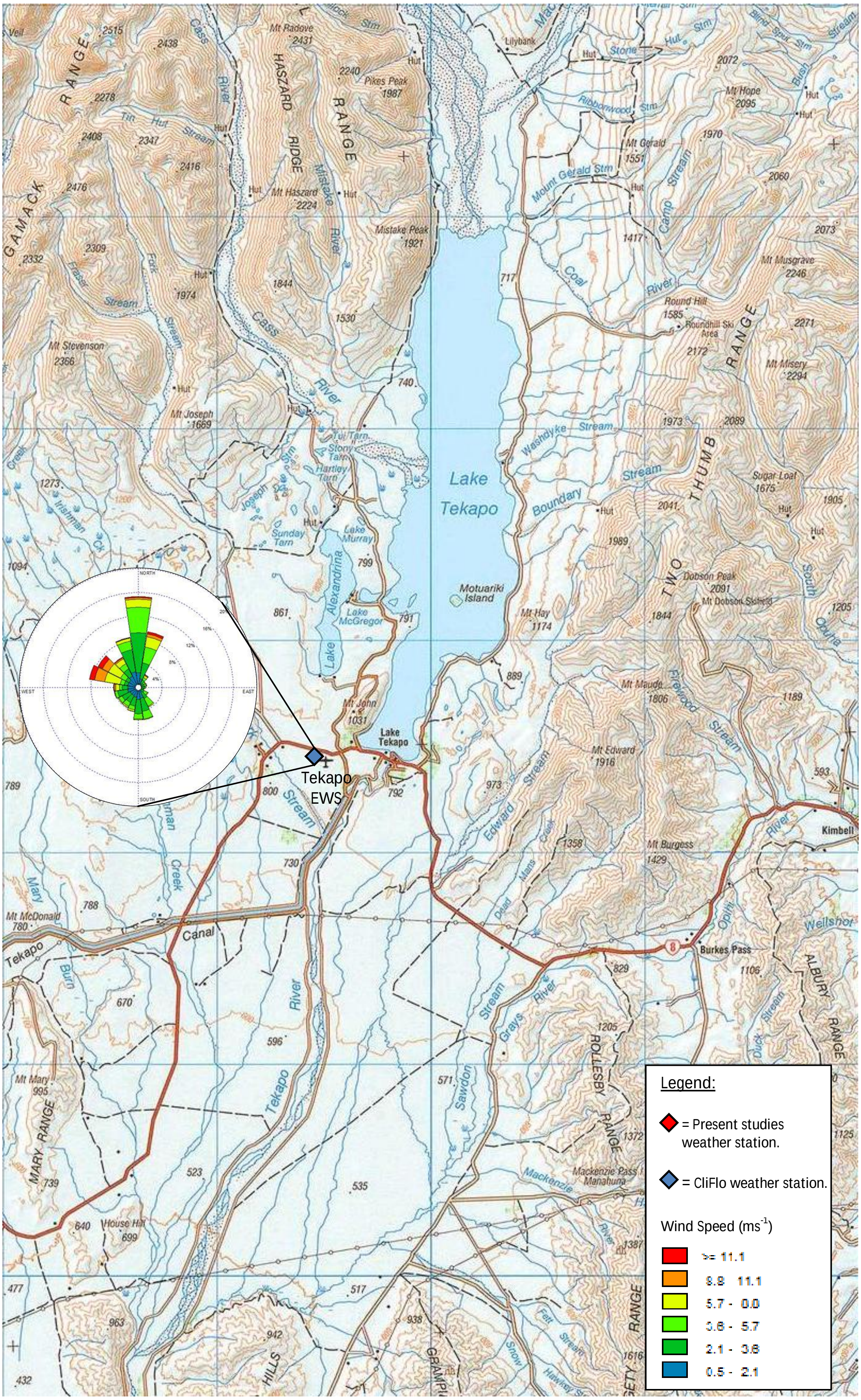


Figure 4.3: The location and associated wind roses for the east, south and CliFlo weather stations near Lake Pukaki (Topomap, 2010). Continued over page.


Tekapo
EWS
Legend:
= Present studies weather station.
= CliFlo weather station.
Wind Speed (ms-1)
>= 11.1
8.8 - 11.1
5.7 - 8.8
3.6 - 5.7
2.1 - 3.6
0.5 - 2.1
NORTH
SOUTH
EAST
WEST
Lake Tekapo
Lake Tekapo
Motuariki Island
Lake Alexandrina
Lake McGregor
Mt John
1031
Tekapo Canal
Tekapo River
Kimbell
Burkes Pass
Mt Joseph
1669
Mt Hay
1174
Mt Edward
1916
Mt Maude
1806
Dobson Peak
2091
TWO THUMB RANGE
HASZARD RIDGE
MARY RANGE
ROLLESBY RANGE
ALBURY RANGE
Cass River
Mt McDonald
780
Mt Mary
995

Kirk's (1988) values are significantly lower because he analysed eleven and a half years of records, whereas this and Allan's (1991) thesis only examined the wind regime around one summer (seasonally windy) period.

At Lake Dunstan Allan (1998) recorded winds >5.7 ms^{-1} 10% of the time and at Lake Coleridge Dawe (2006) measured winds >5.7 ms^{-1} 51.3% of the time. This shows that Lake Dunstan experiences a similar wind climate as Lake Pukaki, whereas Lake Coleridge is a great deal windier. This highlights a point made in Section 3.4.1, explaining the higher wave energy environment at Lake Pukaki. It seems the effective fetch as opposed to the wind speeds is a more important factor in the determining maximum wave heights.

No strong southerly events were recorded in the wind data. A combined total of 97.77% of the wind events exceeding or equal to Force 4 for the east station and 91.28% for the south station were confined to the northwest, north and northeast directions. The remainder of these winds, 2.23% and 8.72%, never surpass Force 4 (8.8 ms^{-1}). Lake Tekapo experiences a similar wind pattern, as all strong gusty winds were restricted to the north, northeast and northwest directions, and were associated with foehn wind events (McGowan *et al.* 1996). For wave hindcasting intentions it is expected that the southern shore will experience a high amount of wave activity generated by strong northerly winds. This is further discussed and analysed in Section 4.5.

In an alpine environment, such as Lake Pukaki, the winds at any particular site are highly localised and the surrounding topography exerts a major control. Careful consideration of the nearby mountainous terrain is required when interpreting the wind and wave conditions of a lakeshore environment (Dawe, 2006). Wind rose plots were created using the programme WRPLOT_View.exe. The wind pattern at both stations is reflective of the surrounding topography as summarised in the wind roses, shown in Figure 4.3. The east and south stations show a prevalence of strong winds from the north and light to moderate winds from the south/southwest. The northerly winds have originated in the Tasman Valley as a result of a topographically channelled northwest airflow. The east station also exhibits an occurrence of light winds from the east. Allan's (1991) wind analysis at Lake Pukaki introduced the occurrence of easterly and north easterly winds at the south eastern end of the lake near the Tekapo B power station. These winds were not discussed in earlier works by Bunting (1977) and Kirk (1988). The presence of the Ben Ohau Range along the western margin of Lake Pukaki acts as a topographical barrier and obstructs the westerly winds. The east station experiences stronger wind speeds than the south station, which never exceed 16 ms^{-1}. The strength of the northerly winds diminishes in a southward direction. As mentioned earlier, the location and exposure of the weather stations could explain the variance in recorded wind speeds.

Verification of the diurnal variation of wind in terms of direction and speed is shown in Figure 4.4 and Figure 4.5. At the east station strong northerly winds (>11.1 ms^{-1}) blow throughout the entire day. Light easterlies begin to develop in the evening after 18:00 and peak between 00:00 and 03:00 at night. A light to moderate southerly occurs between 06:00 and 09:00 when the easterly declines and eventually dies off. The southerly shifts to a south-westerly leading into midday and continues for most of the afternoon, then decreases after 03:00. Allan (1991) states that the southwesterlies are a product of topographic heating resulting in land/water temperature differentials leading to the development of lake breezes. The south-westerly winds recorded by the south station show similar attributes to that of the easterlies recorded by the east station. They both dominate the night time wind regime and decline in the mid morning. These particular winds can be classified as land breezes and are the opposite of what occurs during the day. Northerly winds are predominant throughout the late morning and early afternoon until 06:00 before shifting to a north-easterly. Less infrequent light to moderate northwesterlies also occur throughout the middle of the day. As with the east weather station, the south station also experiences strong gusty northerlies throughout the entire day. This is an important point in terms of wave hindcasting because large storm events created by the north-westerly/northerly winds are not limited to any time during the day.

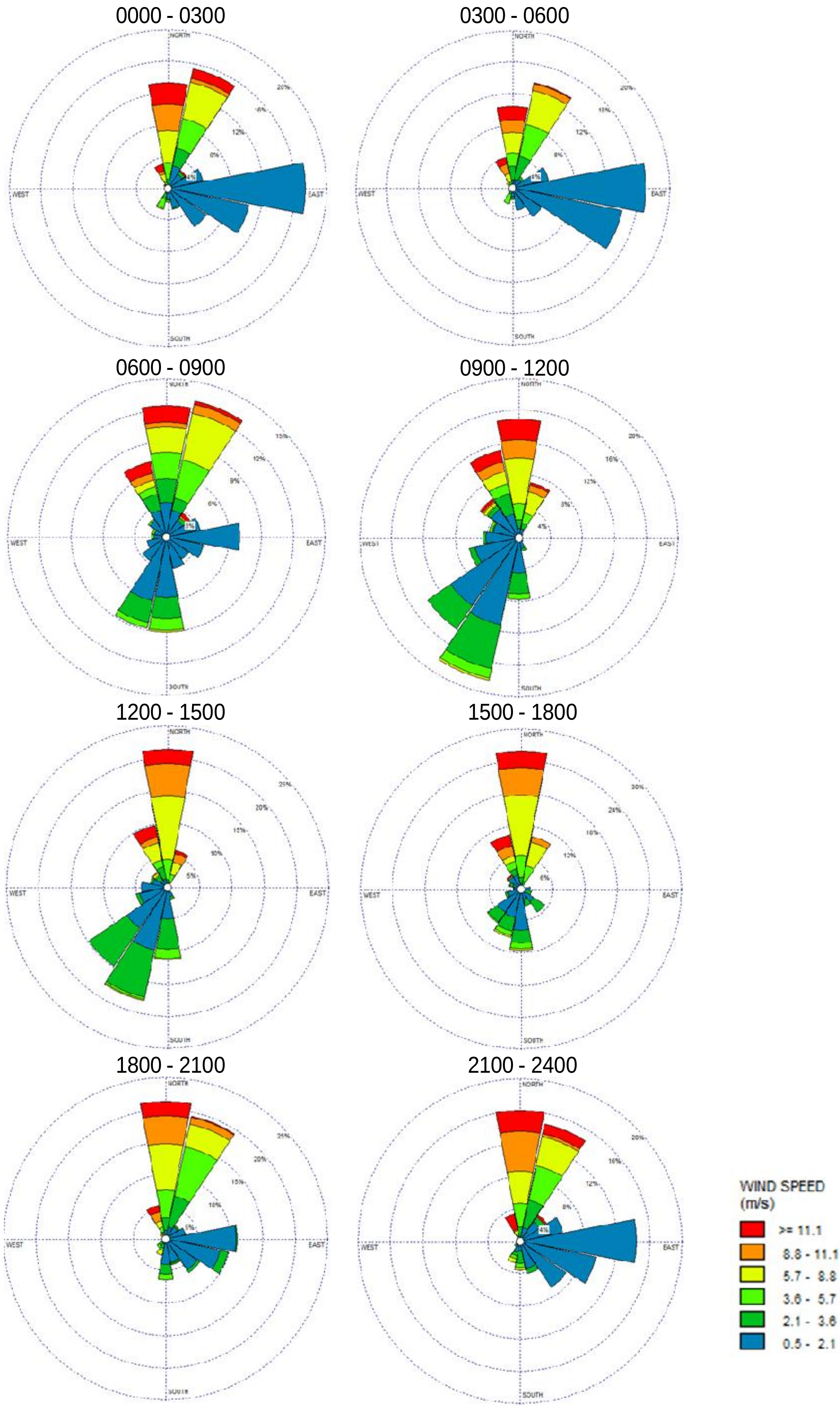


Figure 4.4: Three hourly wind roses for the east weather station from 28/7/10 to 6/2/11.

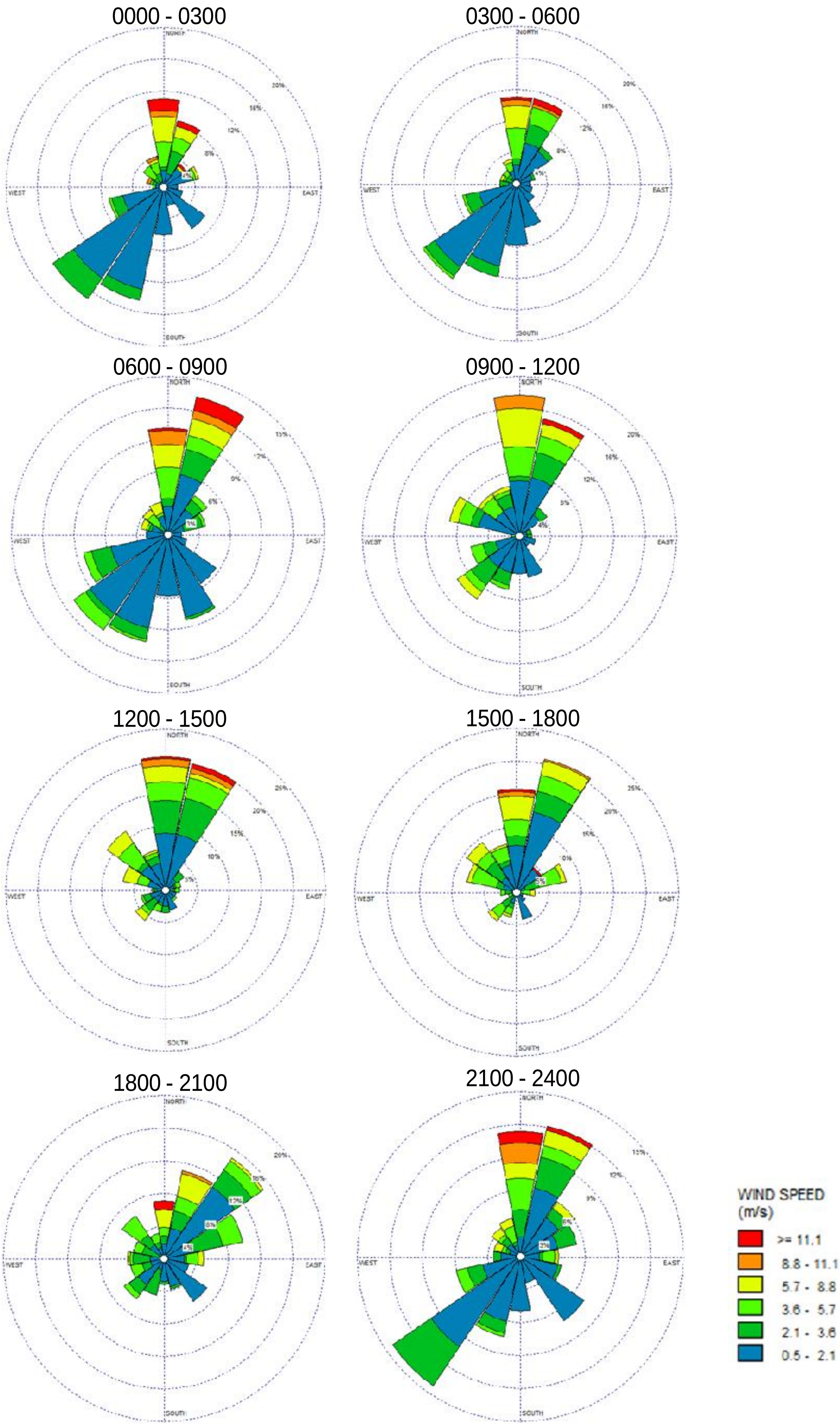


Figure 4.5: Three hourly wind roses for the south weather station from 27/7/10 to 29/10/10.

4.3.3 CliFlo Weather Stations

For wave hindcasting purposes other weather records were obtained so that the wind regime could be examined over the past ten years. Weather records were sourced from CliFlo, which is a web based system that provides access to New Zealand's national climate database, owned and operated by the National Institute of Water and Atmospheric Research (NIWA). An examination of the wind over a decade period illustrates the presence of any localised wind phenomena that occur during a particular season. The data available traces back to early in the year of 2000. The weather stations chosen are in the vicinity of Lake Pukaki (Figure 4.3). It has been stressed that the use of weather records sourced from another location is unreliable for wave hindcasting. Nevertheless, the implementation of these records only reinforces the complex nature of the wind regime for wave prediction.
A comparison of the CliFlo records with the previously discussed east and south station data will determine the suitability of using the CliFlo data for wave hindcasting.

Three CliFlo weather stations have been considered for analysis. Mt Cook EWS is located in Mt Cook village, ~20 km north of Lake Pukaki. Lake Tekapo EWS is situated in the Tekapo aerodrome, 2 km west of Lake Tekapo Township and ~20 km east of Lake Pukaki. Both the Tekapo and Mt Cook sites weather records cover around eight to eleven years of weather records. Lastly, the Pukaki Aerodrome AWS is located, as expected, in the Pukaki aerodrome 3 km north of Twizel and 5 km south of the lake. Unfortunately, the only available data for the Pukaki aerodrome site is from the 12/12/08 to the 6/2/11.

The wind pattern presented in the wind rose for Mt Cook EWS (Figure 4.3) is as would be expected due to its location within the northwest-southeast aligned Hooker Valley. There is a high frequency of light westerly winds that occur throughout the night as katabatic winds. During the day moderate to strong northwesterlies/northerlies and moderate southeasterlies are dominant. Lake Tekapo EWS (Figure 4.3) receives winds from three principal directions. The moderate to strong northwesterlies would have developed within the Tasman Valley, whereas the moderate to strong northerlies are likely to have originated from the Godley and Cass Valley, at the northern end at Lake Tekapo. It is important that these two winds are distinguished, especially for wave hindcasting, because the northerly winds for this station are representative of Lake Tekapo not Lake Pukaki. The third principle wind direction for this site is light to moderate southerlies flowing up from the Tekapo River. The Pukaki Aerodome AWS is situated in an exposed basin and experiences winds from all directions (Figure 4.3). The prevalence of winds arises from the west direction, in particular from the northwest. Kirk's (1988) wind analysis is very similar to that described by the Pukaki Aerodrome AWS site. For his analysis he acquired weather records from Twizel, which explains the similarity as the two sites are located close to one another. The majority of the northwest winds occur during the night and are classed as katabatic winds that originate from the Ben Ohau range in the northwest.

Appropriateness of Weather Records

To justify the applicability of using the CliFlo wind data for wave hindcasting comparisons have to be made to the east and south stations, as they supply representative winds for the lakeshore environment. North-westerly/northerly winds produce the strongest wind speeds at Lake Pukaki. Therefore, a particular northerly wind event on the 12/8/10 is assessed, as detailed in Appendix 4. Specific periods were highlighted and were depicted in either a sudden change in wind speed or direction. The East WS and South WS received a burst of light southerlies just before the northerly wind event occurred. The northerly wind blew for a matter of eight hours at the East WS. As discussed earlier the northerly wind for the Tekapo EWS can be omitted as it is not representative of Lake Pukaki. Mt Cook EWS displays a lighter northwest wind during this time as it is sheltered from airflow through the Tasman Valley and only experiences winds from within the Hooker Valley. Both the South WS and the Pukaki Aerodrome EWS display a lag time before they encounter the northerly winds, though the South WS receives moderate north easterlies before shifting north. All the stations, excluding Mt Cook EWS, showed a shift to the northeast following the northerly event before the winds faded off.

Kirk (1988) and Allan (1991) mention the importance of southerly wind events for causing erosion along the northern half of Lake Pukaki's shoreline. For this reason, a southerly episode on the 7/8/10 was also examined (Appendix 4), as it was the strongest and longest recorded event. As at 17:00 on the 7/8/10 the Pukaki Aerodrome AWS received a moderate westerly wind. During the same hour, the wind shifts to the southwest for the South WS and to the south for the East WS. This southerly event must originate south of the Ben Ohau range and bend around northwards to be channelled up Lake Pukaki. It takes an additional three hours for the Tekapo EWS to experience winds from the southwest. The south-westerly/southerly winds abate for the South WS and the East WS from 5:00 onwards. Throughout the entirety of the southerly event the Mt Cook EWS displays a range of wind speeds and directions and shows no uniformity with the other stations, apart from a moderate southerly channelled up the Hooker Valley at 19:00.

The suitability of the CliFlo stations used for wave hindcasting will be discussed in the following section. Due to the locations of the CliFlo stations it will be required that correction factors be applied to them, these are implemented in Section 4.4.

4.3.4 Discussion of Wind Results and CliFlo Station Comparison

The surrounding mountainous terrain exerts a major influence on the wind pattern at Lake Pukaki. The north-south orientation directs airflow down the long axis of the lake. Winds originating from the northerly direction were more pronounced and occurred throughout the entire day. According to Kirk's (1988) lower threshold for wave generation, winds exceeding 5.7 ms^{-1} eventuate 97.77% and 91.28% from the northwest, north and northeast directions, for the east and south stations respectively. This does not completely exclude the southerlies for examination as recorded wind speeds have surpassed this lower threshold. The easterly night time land breezes would be unable to produce sufficient wave heights to cause erosion to the western shore as winds from this direction never exceed 2.1 ms^{-1}. The east station experienced stronger winds than the south station due to their site locations and their exposure to the lake wind field. The oscillation of lee waves could also explain the difference in recorded wind observations for both of these stations (Allan, 1991). Stronger north-westerly/northerly winds may be constrained to the north end of Lake Pukaki.

As with the previous stations, the topography was found to have an influential effect on the wind pattern for the CliFlo stations. After comparison with the east and south station wind records, several characteristic attributes of the airflow have been introduced. Mt Cook EWS is not exposed to the full force of the northerly winds entering the northern end of Lake Pukaki as it is confined to the airflow within the Hooker Valley. This explains the lower magnitude of wind speeds in contrast to the east station. The Tekapo EWS is susceptible to northwest winds arising from the Tasman Valley. Generally, the occurrences of northwest winds at Tekapo EWS coincide with northerly winds at Lake Pukaki. However, the northerly and southerly winds for Tekapo EWS are representative of Lake Tekapo and Lake Alexandria. The Pukaki Aerdrome AWS demonstrates good correlation to the south stations wind pattern, especially in terms of northerly winds. On the other hand, during a southerly event at Lake Pukaki, the Pukaki Aerodrome AWS usually experiences a westerly wind.

Overall, for wave hindcasting, more emphasis will be placed on data from the east station data because it recorded for a longer duration and was situated near the shoreline, well exposed to the lake winds. Due to its location, no corrections need to be applied to the wind data in terms of height or distance from the lake. The south station wind records will not be omitted from analysis, though height corrections are required and these are calculated in Section 4.4. Only a few specific wind directions from particular CliFlo sites were recognised as being suitable for the purposes of estimating storm events since 2003. Winds from the northwest direction for the Tekapo EWS were deemed appropriate as were the northerly events at the Pukaki Aerodrome AWS. The southerly winds for this latter site were less relevant. Mt Cook EWS wind observations were deemed inapplicable, as the station is not exposed to the full force of the accelerated winds originating at the north end of Lake Pukaki in the Tasman Valley. The Tekapo EWS northerlies and southerlies are site specific and not representative of Lake Pukaki.

4.4 Adjustment of Wind Data

Winds are susceptible to frictional effects when they develop within the boundary layer, generally beneath 1000 msl, and as a result are dependant upon elevation above the mean surface, roughness of the surface, and air-temperature difference (CERC, 1984a). For wave hindcasting the wind speeds therefore require adjustment to account for the difference in frictional forces experienced over land and water. The LAKEWAVE program automatically adjusts the wind data if it is required. These processes are presented by CERC (1984) and are outlined below.

The standard height for a wind anemometer for wave hindcasting/forecasting is 10 m above the water level (Resio *et al.* 2002). The base of the east weather station was 539.82 msl, and the height of the anemometer was ~542.32 msl. The east weather stations anemometer was approximately 10 m above a lake elevation of 532 msl, the upper end of the operating range. No corrections are therefore required for the east station as it is representative of lake winds. Due to the large operating range of ~14 m a wind anemometer will not remain at a still height of 10 m above the water surface throughout the entirety of recording. For the purposes of this study it was assumed that no further height correction is needed for the east station. From a topographical map the south station was estimated to be ~580 msl. As it is not situated at the desired height of 10 m above the lake level, wind speeds need to be adjusted therefore using Equation 4.3:

$$U_{10} = U_o \left(\frac{10}{Z}\right)^{\frac{1}{7}} \qquad \text{Equation 4.3}$$

Where U_{10} is the adjusted wind speed, U_o is the observed wind speed and Z is the height of the instrument above the water level in metres. The wind data for the Tekapo EWS (762 msl) and the Pukaki Aerodrome AWS (473 msl) also had to be corrected to the 10 m reference height.

A stability correction factor was also required for adjustment of the wind data. This takes into account the temperature difference between the air and water surface:

$$\Delta T_{AW} = T_A - T_W \qquad \text{Equation 4.4}$$

Where ΔT_{AW} is the air-water temperature differential (ºC), T_A is the air temperature (ºC) and T_W is the water surface temperature (ºC). The amplification ratio (R_T) is defined by Figure 3-14 in CERC (1984a, 3-31) for the effects of air-water temperature differentials. When ΔT_{AW} is zero the boundary layer has neutral stability and no adjustment is needed for the wind data. If ΔT_{AW} is negative, the boundary layer is unstable, enhancing wave growth. A positive ΔT_{AW} means that conditions are stable and wave growth is less effective (CERC, 1984a).

The effective wind speed (U_A) is accordingly defined as:

$$U_A = U_{10}\, R_T \qquad \text{Equation 4.5}$$

The amplification factor has been calculated for four separate events at Lake Pukaki, shown in Table 4.2. For the purposes of comparing measured and hindcast wave data, these events were chosen as they coincide with dates when wave recording was carried out. Air temperature data was supplied by the east station and water temperature figures were derived from the RBR XR-620 wave recording device. All the events display stable conditions in terms of wave development. These values approach those of Allan (1991), who obtained an amplification factor of 0.94, determined from an average air/water temperature difference of 1.35°C at Lake Pukaki. Due to the absence of temperature records Kirk (1988) used an amplification factor of 1.1 for wave hindcasting, which is suggested by CERC (1984a), promoting unstable conditions. Allan (1991) states that an R_T factor equal to 1.1 should not be implemented for South Island lakes when there is a deficiency of ΔT_{AW} data. A value of 1.1 is likely to over-estimate wave characteristics and he recommends applying a more conservative figure of 0.94.

For wave hindcating purposes it is preferable that overwater wind records are utilised. When these are unavailable wind observations acquired from a climate station that is situated inland from the water source may be used. However, corrections need to be applied to the wind data to justify the difference in frictional forces experienced over water and land. Figure 3-15 in CERC (1984a, 3-31) corrects the observed wind speed to an over water wind speed. For this study, adjustment of the east and south weather station is not necessary because both of the stations are located near Lake Pukaki's shoreline. The wind speeds for the Tekapo EWS and Pukaki Aerodrome AWS do need adjustment as they are situated ~20 km east and ~5 km south of Lake Pukaki respectively. The techniques, outlined by CERC (1984a) for adjusting wind speeds are all based on studies conducted on the Great Lakes. Wave estimates on the Great Lakes have been developed largely because of the existing knowledge of the passage of storm systems, the size of the lakes and the flatness of topography (Allan, 1991). Due to the apparent differences between the Great Lakes and alpine lakes in New Zealand, previously discussed in Section 1.2.3, caution should be taken when applying these methods to an alpine lake such as Lake Pukaki. The appropriateness of utilising these techniques for wave hindcasting will be analysed in Section 4.6.3.

Table 4.2: Air-water temperature differentials (ΔT_{AW}) and amplification ratios (R_T) for events coinciding with wave recording at Lake Pukaki. Derived from the air temperature (T_A) and the water surface temperature (T_W).

Event Date	T_A (°C)	T_W (°C)	ΔT_{AW} (°C)	R_T
12/8/10	10.06	7.27	2.79	0.89
18/12/10	22.22	17.18	5.04	0.85
15/1/11	18.38	15.58	2.80	0.89
6/2/11	23.91	15.54	8.37	0.83

4.5 Estimation of the Lake Pukaki Wave Environment

The twelve study sites, mentioned previously in Chapter 2 and presented in Figure 2.9, have all been selected to represent the wave climate at Lake Pukaki. Predicted wave hindcast properties were carried out for each of the twelve sites, during the study period from July 2010 to February 2011. Five of the events utilised for wave hindcasting involved strong winds originating from the north. The strongest northerly event was recorded on the 24/09/10 by the east weather station, which endured for five hours with an average wind speed of 24.11 ms^{-1} and direction of 340°. The LAKEWAVE wave statistics predicted from this storm event are presented in Table 4.3. The other four northerly hindcasted episodes coincide with when the XR-620 was deployed (12/08/10, 18/12/10, 15/01/11 and 6/02/11) and are used for the purposes of correlation, which are analysed in Section 4.6. Due to the lower magnitude and frequency of southerly winds only one southerly wind event was considered for analysis. The east station data was applied for all six cases.

According to the modelled results shown in Table 4.3, wave size amplifies in proportion to the increase in fetch length. The largest waves for the 24/09/10 event were predicted to occur along the southern shoreline at Site 8, with H_s = 2.35 m, H_b = 3.03 m and T = 5.6 s. The wave breaker height (H_b) considers wave shoaling and refraction.

$$H_b = H_s K_S K_R \qquad \text{Equation 4.6}$$

$$H_b = H_s \, 1.14^2 \cos(\alpha_b)^{0.5} \qquad \text{Equation 4.7}$$

Where K_S is the shoaling coefficient, K_R is the refraction coefficient and α_b is the breaker approach angle. α_b is measured in degrees from shore normal to the beach face. A positive value represents wave approach from the right side facing lakeward and a negative value constitutes wave approach from the left side, which is distinctive of the eastern and western shorelines respectively for a northerly wind event.

Wave refraction is maximised when the shoreline is orientated parallel to the wave approach or there is interference from nearby embayments or alluvial fans and is represented by low H_b values compared to the associated H_s. The eastern and western shores are shown to experience a large proportion of wave energy at oblique angles. This will eventually lead to the formation of strong longshore currents for these sites. In contrast, the wave approach for the southern shore is predominantly perpendicular to the beach face, resulting in larger H_b values as the waves undergo minimal refraction.

The strongest recorded southerly wind averaging 7 ms^{-1} on the 7/08/10 was predicted by LAKEWAVE to produce the largest waves at Site 1, with H_s = 0.47 m, H_b = 0.34 m and T = 2.7 s. Sites 7-10 are unaffected by wave energy during a southerly wind event due to their location along the southern shore.

Table 4.3: LAKEWAVE predicted wave statistics for all study sites at Lake Pukaki during a northerly wind event averaging 24.11 ms^{-1}, which lasted for five hours on the 24/09/10. H_s is the deepwater significant wave height, H_b is the breaker height, T is the wave period and α_b is the breaker approach angle.

Site	H_s (m)	H_b (m)	T (s)	α_b (°)
1	1.05	0.74	3.5	73
2	1.32	1.68	4	17
3	1.43	1.47	4.2	51
4	1.54	1.3	4.4	65
5	1.63	1.08	4.5	75
6	1.85	1.34	4.8	72
7	2.05	2.38	5.1	37
8	2.35	3.03	5.6	-10
9	2.32	3.01	5.5	-5
10	1.38	1.48	4.3	-47
11	1.67	0.9	4.6	-80
12	0.73	0.58	2.9	-68

North/north-westerly wind events recorded by Tekapo EWS and Pukaki Aerodrome AWS have been identified as suitable for wave hindcasting at Lake Pukaki. The utilisation of this wind data, dating back to early 2003, is to investigate the possibility of a storm event coinciding with a high lake level. The affect of lake level elevation on wave height was discussed in Section 2.3.2. On the 8/2/09 Tekapo EWS recorded wind speeds averaging 11.66 ms^{-1} producing a maximum potential H_s = 0.94 m and T = 3.7 s, when Z = 230 m. The same wind data was recalculated applying no height adjustments (i.e. Z = 10 m). A H_s = 1.41 m and T = 4.4 s were determined, representing a 50% increase in wave height, which is significant. During a separate event, on the 9/1/09, the Pukaki Aerodrome AWS recorded winds speeds that averaged 10.9 ms^{-1}, giving rise to a maximum potential H_s = 1.8 m and T = 4.9 s, when Z = -59 m. The same method was applied, as with the Tekapo EWS northerly event, whereby the wind data was adjusted to the 10 m reference level. A subsequent H_s = 1.33 m and T = 4.3 s were ascertained, indicating a 26.1% reduction in wave height. The lake level was ~532.2 msl on the 8/2/09 and the 9/1/09, and an assumed R_T of 0.89 (ΔT_{AW} = 2.8°C) was applied for both cases. The resultant wave height is highly influenced by the elevation adjustment determined by the "1/7 Rule" (Equation 4.3) because of the substantial elevation difference of the Tekapo EWS and Pukaki Aerodrome AWS in relation to Lake Pukaki's water level. Resio *et al.* (2002) state that the "1/7 Rule" should only be used within an elevation range of about 8-12 m above the water surface in near-neutral conditions (ΔT_{AW} = 0-3°C). The analysis of the northerly wind events pertaining to the Tekapo EWS and the Pukaki Aerodrome AWS have not been considered further due to the unreliability of the resultant wave heights.

LAKEWAVE determines the time it takes for a stable fully arisen sea (FAS) state to build, which is dependant upon the fetch, wind speed and direction. When the wind event duration exceeds the time for a FAS it becomes fetch limited, whereas the condition before a FAS is reached is referred to as duration limited. Waves do not continue to grow in size, obtaining their maximum height, when FAS has developed. Table 4.4 demonstrates the required duration for FAS to develop at Lake Pukaki from a given wind speed, for the longest fetch situated on the southern shore during a northerly wind event. Wave growth on Lake Pukaki is predominantly fetch limited. Winds that exceed 8.8 ms^{-1} tend to endure for at least four hours, the time required to develop FAS.

Table 4.4: Time required for the development of a FAS during a northerly wind event along the southern shoreline according to LAKEWAVE.

Wind Speed (ms^{-1})	Time for FAS (hours)	H_{so} (m)	T (s)
5	5.6 – 5.7	0.33 – 0.35	2.3 – 2.4
10	3.8 – 3.9	0.77 – 0.82	3.4 – 3.5
15	3.1 – 3.2	1.26 – 1.35	4.2 – 4.4
20	2.6 – 2.7	1.8 – 1.93	4.9 – 5.1
25	2.3 – 2.4	2.37 – 2.53	5.6 – 5.7
30	2.1 – 2.2	2.97 – 3.17	6.1 – 6.3

Recapping on an assumption of the LAKEWAVE model, wave hindcasting requires wind speeds that are constant across the entire fetch length and do not deviate by more than 2.5 ms^{-1}. For the assessed wind event on the 24/09/10 wind speeds at the east station were at least two times greater than the winds at the south station. As wind speeds are not constant across the entire fetch, it is assumed that wind/wave energy exchanges are also not occurring across the entire fetch. Therefore, the likelihood of a wave growing to H_s = 2.32 m at Site 8 is doubtful. Allan (1991) hypothesised that wind waves will often become decoupled from the generating area, which will reduce the potential size for wave propagation, leading to the development of swell waves. He believed that the variation in observed wind speeds across the long axis of the lake was due to the occurrence of north/north-westerly foehn winds oscillating in the Pukaki Valley. This explains why stronger northerly winds may be restricted to the northern half of the lake.

Both Kirk (1988) and Allan (1991) have made attempts at wave forecasting/hindcasting on Lake Pukaki. Based on his wind analysis, Kirk (1988) predicted that winds of 20–23 ms^{-1} produce a H_S of 3.55 m and a T of 6.75 s. Allan (1991) incorporated the same data set of Kirk (1988) but included a stability factor (R_T) derived from air/water temperature differences and calculated an H_S which showed a 33.2% reduction in height. He suggested an upper limit of wave height, for the longest fetch of 30.8 km, is ~2.7–3.04 m, in winds of 30.07–33.89 ms^{-1}. These latter values obtained by Allan (1991) correlate well with what was predicted in this study using LAKEWAVE, referring to Table 4.4. The present author believes that a wave height (H_S) exceeding 3 m is not likely to occur at Lake Pukaki. Average hourly wind speeds recorded during this study and obtained from CliFlo stations since early 2000 have never surpassed 25 ms^{-1}. The inconsistency of winds remaining constant across the entire fetch also reinforces a reduction to the maximum wave height initially estimated by Kirk (1988) and Allan (1991). An upper limit H_S of ~2 – 2.3 m is more suitable for Lake Pukaki.

4.6 Predicted vs. Measured Wave Statistics

An initial aim of this study was to ascertain the validity of using the LAKEWAVE program to predict wave characteristics at Lake Pukaki. The LAKEWAVE hindcasted data is correlated with wave statistics measured by the XR-620. The number of cases that were used in correlation is not large, ranging from n = 37 at Site 8 to n = 7 at Site 7. All of the events used for analysis were recorded when a gusty north/northwest wind prevailed. The wind directions were confined between 340° to 20°.

4.6.1 Correlation of Measured and Modelled Wave Height

Measured wave heights obtained by the XR-620 and predicted LAKEWAVE wave heights were correlated by way of linear regression analysis. The deepwater significant wave height (H_s) is the parameter being compared. Correlation of the measured H_s with the breaker height (H_b), estimated by LAKEWAVE, proved to be less comprehensive than with the estimated H_s. Results for each of the three study sites are presented in Figures 4.6, 4.7 and 4.8.

Examination of the correlations between measured H_s and predicted H_s (Figures 4.6, 4.7 and 4.8) indicate that the LAKEWAVE model is inconsistent at specifying the wave environment for Lake Pukaki. There is prolific variability shown after comparing the regression plots, as the strength of correlations range from $R^2 = 0.48$ to $R^2 = 0.83$. Overall, the LAKEWAVE model tended to over predict H_s.

Both Site 7 and Site 8 are situated on the southern shoreline and are exposed to the longest fetches during a north/northwest wind event, thus receiving the largest wave heights. The wave data gathered at Site 7 (Figure 4.6) and at Site 8 on the 18/12/10 (Figure 4.8) correlates thoroughly with the predicted wave heights. A $R^2 = 0.83$ and $R^2 = 0.77$ was obtained for the 12/8/10 and 18/12/10 events respectively. This essentially indicates that 83% and 77% of the data is able to be explained.

It is interesting to note the difference in correlation of the two wave events analysed at Site 8 (Figure 4.8). The predicted and measured H_s for the 15/01/11 event do not correlate well ($R^2 = 0.48$) as the residuals suggest a negative relationship ($R = -0.69$). The 15/01/11 event experienced the strongest winds and largest waves out of all the occasions when wave recording was carried out. However, the LAKEWAVE model estimated that FAS would occur faster than expected. The LAKEWAVE program took into account an increase of wind speeds at 9 am from 4 ms^{-1} to 10 ms^{-1} and predicted FAS to develop at midday with H_s reaching 0.99 m and steadily decreasing to 0.84 m in the early evening. In contrast, the measured H_s increased throughout the afternoon and evening from 0.34 m to 1.07 m.

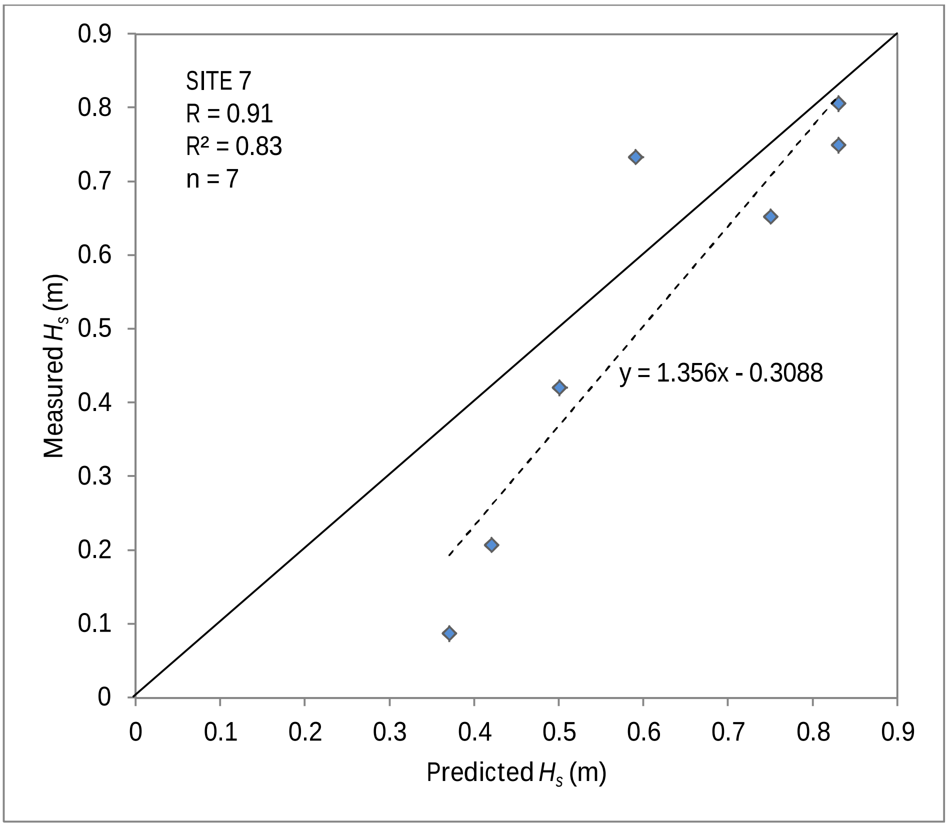


Figure 4.6: Regression of predicted significant wave height to measured significant wave height for Site 7 on 12/08/10.

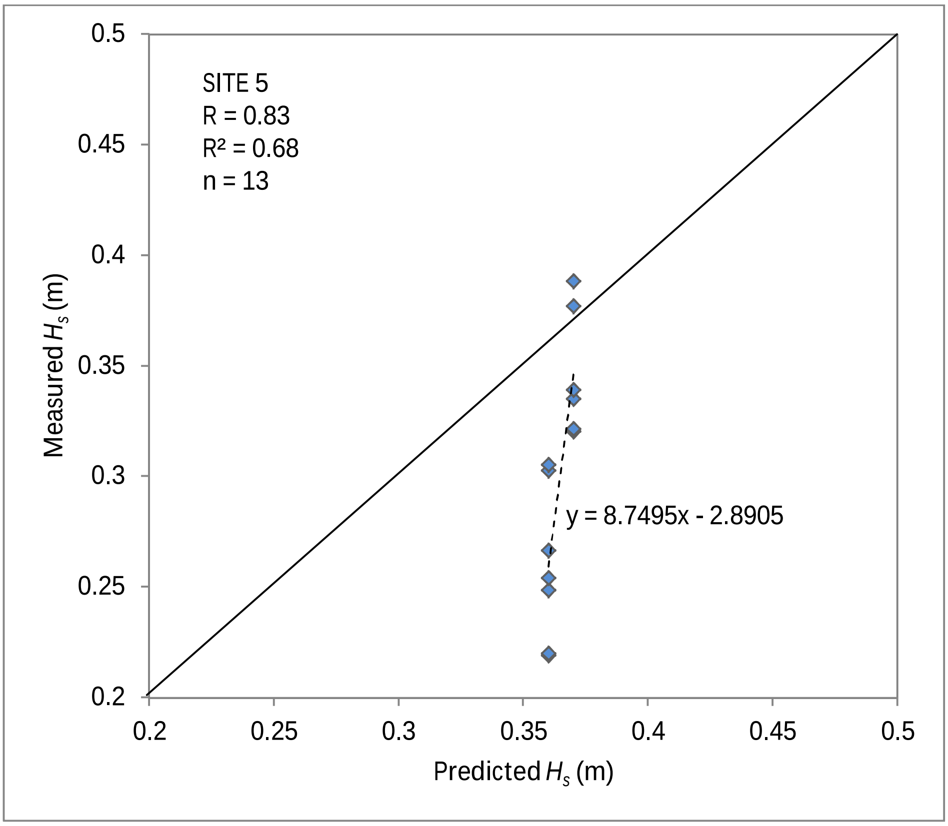


Figure 4.7: Regression of predicted significant wave height to measured significant wave height for Site 5 on 6/02/11.

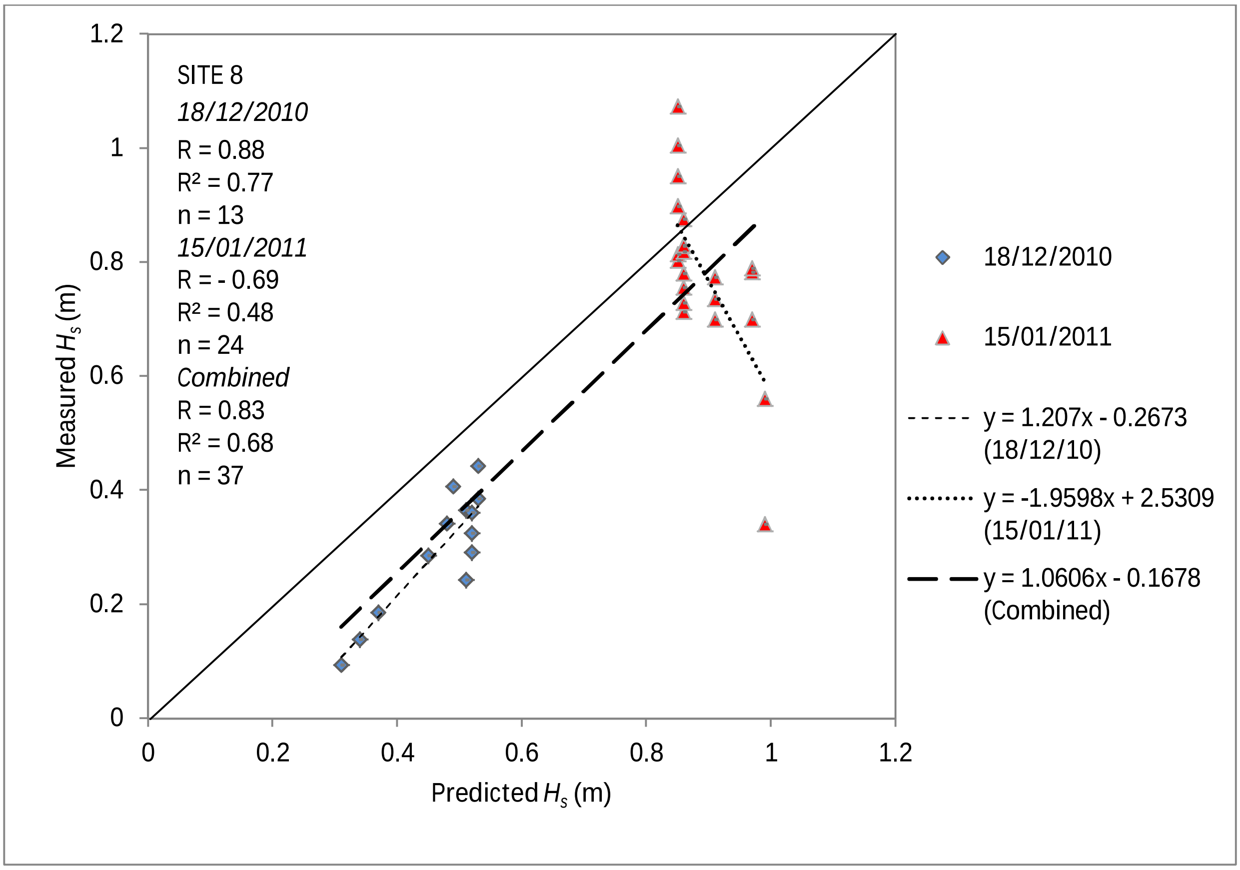


Figure 4.8: Regression of predicted significant wave height to measured significant wave height for Site 8.

There is an evident delay of wind speeds recorded by the east station and what is experienced along the southern shoreline on the 15/01/11. This finding further highlights the degree of variability of winds along the north-south axis length of Lake Pukaki.

A reasonable correlation (R^2 = 0.68) is identified at Site 5 on the 6/02/11 (Figure 4.7). Site 5 is well represented in terms of wind conditions as it is located on the eastern shoreline adjacent to the position of the east station. Therefore, any discrepancies involving the correlation of predicted and measured wave statistics at this site is not due to wind data utilised for the LAKEWAVE model. On the 6/02/11 the LAKEWAVE model predicted FAS to occur much sooner than what was observed and measured in the field. The predicted H_s values remained constant (0.36 m to 0.37 m) because according to LAKEWAVE, FAS had already developed while the measured H_s ranged between 0.22 and 0.39 m.

4.6.2 Correlation of Measured and Modelled Wave Period

Linear regression analysis was also carried out to correlate measured wave period with the modelled wave period. The results of regression analysis for the wave period are summarised in Table 4.5. T_z obtained by the XR-620 was used to correlate with the modelled wave period. The use of T_s instead of T_z did not result in any improvement for the regression analysis. The accurateness of the LAKEWAVE model is very variable at predicting wave period statistics at Lake Pukaki. 75 % of the residuals were overestimated by the LAKEWAVE model. According to Table 4.5, similar correlations exist that were noticed in section 4.6.1 with the wave height regression analysis. The predicted and measured wave periods compare well for the 12/08/10 and 18/12/10 wave events, whereas the 15/01/11 and 6/02/11 events display a relatively poor correlation.

Table 4.5: Regression analysis of modelled (LAKEWAVE) wave period against measured wave period, obtained by the XR-620.

Location	Date	R	R^2
Site 7	12/08/10 (n = 7)	0.90	0.80
Site 8	18/12/10 (n = 13)	0.90	0.81
Site 8	15/01/11 (n = 24)	-0.50	0.25
Site 5	6/02/11 (n = 10)	0.23	0.05

The predicted and measured wave statistics do not generally match up for each twenty minute interval as the LAKEWAVE model predicts FAS to develop earlier than what has been measured in the field. However, over the whole recording period for each individual event the maximum values obtained from the LAKEWAVE model and the XR-620 do correlate well. Table 4.6 contains the modelled and measured maximum wave height and wave period values for each of the four analysed events. As expected, the maximum values do not always occur at the same time. Acknowledging that there is only minor variation with the figures presented in Table 4.6 the LAKEWAVE model can be considered reliable at predicting the maximum H_s and T values for each of the four specified dates.

Table 4.6: Maximum wave height and wave period values acquired by the LAKEWAVE program and the XR-620. * denotes values that occurred at the same time.

Date	Wave Height (m)		Wave Period (s)	
	Predicted (H_s)	Measured (H_s)	Predicted (T)	Measured (T_z)
12/08/10	0.83*	0.81*	3.5*	3.7*
18/12/10	0.53*	0.44*	2.9*	2.6*
15/01/11	0.99	1.07	3.8	3.8
6/02/11	0.37*	0.39*	2.4	2.8

4.6.3 Validity of the LAKEWAVE Model

After reviewing the degree of correlation between predicted and modelled wave statistics a number of calibration issues have been highlighted for the LAKEWAVE model. In general, LAKEWAVE was found to over predict wave height and wave period at Lake Pukaki. A similar pattern was discovered by Allan (1998) when he correlated measured waves at Lake Dunstan with predicted waves determined by the NARFET model. As mentioned previously in Section 4.2, LAKEWAVE is based on the NARFET model developed by Smith (1991). Allan (1998) found that NARFET over predicted the wave height and under predicted the wave period at Lake Dunstan, especially for sites located along the axial shore of the lake.

LAKEWAVE estimates FAS to occur much sooner than what was measured in the field. Wave development within the LAKEWAVE model ceases when these conditions prevail. This phenomenon is justified on the 15/01/10 at Site 8 and on the 6/2/11 at Site 5. The measured wave heights continue to increase whereas the modelled values stay constant. The time required for FAS to develop on Lake Pukaki is on average ~27% longer than what is implied by the LAKEWAVE model.

It is evident that the source of wind data is essential in terms of accurate wave prediction. The discussed discrepancies that exist within the LAKEWAVE model are not entirely due to the model itself. There is a heavy reliance on the wind data supplied by the east station and that it is supposed to represent the entire wind field for Lake Pukaki. It was noted in Section 4.3 that complex wind flows exist within the Pukaki valley and that a thorough understanding of the surrounding topography is required to comprehend this. LAKEWAVE is unable to take into account variable and gusty winds that are accelerated down valleys as it requires a constant wind blowing across the entire fetch. This may explain why the model predicts FAS to occur sooner than what was measured in the field. Allan (1998) remarked that hindcasting models often struggle to estimate wave characteristics in fetch limited alpine environments where the winds are subject to topographical streaming.

Aside from the apparent problems, the LAKEWAVE predicted wave height and wave period correlated well when compared to the measured wave data on the 12/08/10 at Site 7 and on the 18/12/10 at Site 8. The maximum H_s and T values estimated by LAKEWAVE have been found to compare well with measured wave statistics, although they do not always occur at the same time. This finding reflects the reliability of LAKEWAVE being able to estimate maximum wave parameters over the course of a specific period or event. Predicted daily maximum wave statistics will be implemented for calculations for design purposes in Chapter 6. Most design equations that take into consideration wave parameters such as H_s or T, require the maximum associated values to model the worst-case scenario.

This section has highlighted some areas where there is a lack of calibration with the LAKEWAVE wave hindcasting model to measured wave variables. To warrant the accuracy of the LAKEWAVE model and its utilisation for wave prediction at Lake Pukaki and other South Island alpine lakes, a more distinguished time series of wave data is required. The paucity of the number of samples used for correlation highlights an area for further research.

4.7 Conclusion

Analysis of the wind regime on Lake Pukaki has identified two principle wind directions for wave generation. The influence of topography channels airflow down the long north-south axis of the lake. Waves that cause the most significant erosion along the shoreline are associated with winds primarily from the northwest/north/northeast sector originating from the Tasman Valley. Modelling results from this study indicate that frequent, high magnitude northerly winds produce maximum wave heights along the southern shore. The eastern and western shores predominantly experience wave energy at oblique angles, leading to the development of longshore currents. Winds from the south/southwest are classed as of secondary importance in terms of wave development.

This analysis further emphasises the importance of utilising weather records from a site representative of the lake wind field. The accuracy of hindcasted wave heights decreases when having to apply corrections factors to account for differences in elevation and frictional affects occurred over land. It is difficult to adjust wind data accordingly in a complex alpine environment such as Lake Pukaki, employing techniques originally applied to the Great Lakes in Northern America. The east weather station data has been implemented for the majority of wave hindasting, simply because no adjustments were necessary, apart from the use of a stability correction factor (R_T). An R_T of 0.83-0.89 was applied, promoting stable conditions, resulting in wave growth being less effective.

The LAKEWAVE model requires a constant wind blowing across the entire fetch. Accepting this assumption has proven to be difficult due to the existence of oscillating airflow along the Pukaki Valley. Allan (1991) introduced the concept of swell conditions occurring on Lake Pukaki largely because wind/wave exchanges do not happen across the entire fetch, and that waves may become decoupled from the wind and propagate outside the wave generating area. The ability to accurately predict waves in an alpine lake setting therefore requires a thorough understanding of the wind regime, including knowledge of airflow characteristics in complex terrain (McKendry *et al.* 1986; McGowan *et al.* 1996).

Upon correlation with measured wave data the LAKEWAVE model tended to over predict wave characteristics (H_s, T) and the time required for a FAS to develop. The latter is likely due to the inconsistency of wind strengths measured along the north-south axis of the lake. It has been confirmed, though, that the model is suitable for estimating maximum wave statistics. The small sample size (n = 57) may not be representative of the entire wave spectrum at Lake Pukaki, but portrays the upper end of it. It was intended as part of this study to measure waves produced by storm events to examine their maximum erosive potential. Knowledge of maximum wave statistics is required for use in design and performance equations for shoreline structures, which is covered in Chapter 6. A larger sample size would have been preferable, which has revealed an area for further research.

Chapter 5

EXTREME CONDITIONS

5.1 Introduction

The purpose of this chapter is to assess the frequency and occurrence of extreme conditions that have the potential of developing at Lake Pukaki. Of high importance are storm events that coincide with high lake levels. It is during these conditions that quite severe erosion can take place along the backshore (Allan, 1998). In terms of shoreline management, identifying the maximum elevation waves are able to act on the shore profile leads to a better understanding of design specifications required for coastal protection structures. Having the ability to accurately estimate maximum wave statistics also results in a more cost-effective design of coastal structures (Dawe, 2006). Coastal protection works operate at a high cost in terms of maintenance and repair when extreme conditions are not well understood.

Over the study period (July 2010 – February 2011), the summer months, in particular January 2011, were revealed to demonstrate high lake levels and a high frequency of strong northerly winds. The LAKEWAVE model has been utilised to examine the wave environment under these specified conditions for several study sites (Figure 2.9) located along Lake Pukaki's shoreline. It was shown in Section 4.6.3, that the LAKEWAVE model is reliable at predicting maximum daily wave statistics.

In this chapter, maximum wave runup, wave energy and wave power estimates are evaluated using hindcasted wave data, provided by LAKEWAVE, under extreme conditions. Longshore sediment transport rates are also briefly discussed and the application of using the LEXSED formula, developed by Dawe (2006), for the Lake Pukaki setting is appraised.

5.2 Wave Runup Elevations at Lake Pukaki ($R_{2\%}$)

The maximum vertical extent of wave swash on a beach above the still water level is known as the runup. The upper limit of wave runup is an essential parameter for determining the active portion of the beach profile (Smith, 2003). According to Allan (1998, p 192):

> *"Identification of the total runup elevations on the shore is extremely important for the purpose of shoreline management. This is because it may be used to estimate the maximum shoreward extent at which waves can cause erosion of infrastructure backing a particular shore."*

Since this study is concerned with extreme runup elevations a parameter $R_{2\%}$ has been utilised, which is the runup that only two percent of the wave runup values observed will reach or exceed. This model was developed by Holman (1986) and has been successfully applied by Kirk *et al.* (1996, 2000) and Komar *et al.* (1996). Holman (1986) determined wave runup to be a function of beach slope, wave height, wave period and the irregularity of the shoreline:

$$R_{2\%} = C\left(\frac{g}{2\pi}\right)^{0.5} \beta {H_s}^{0.5} T \qquad \text{Equation 5.1}$$

Where β is the beach slope in degrees and C is a coefficient that depends upon the roughness and permeability of the slope. Equation 5.1 also takes into account the vertical elevation attained due to wave induced setup. C is determined by site conditions and varies between 0.83 for a rocky slope to 1.5 for a smooth slope. A C value of 0.9 has been applied for analysis as Kirk *et al.* (1996) used this figure for evaluating runup heights at Lake Hawea, which demonstrates very similar site and sedimentary conditions to Lake Pukaki.

5.2.1 Extreme Wave Runup Analysis: January 2011

Daily maximum wave runup ($R_{2\%}$) and total water elevations have been evaluated for six sites situated along Lake Pukaki's shoreline, during the month of January 2011. Section 2.3 identified that over the fieldwork programme of this thesis the highest lake levels occurred during January 2011 and that this month is the third windiest behind September and December 2010, according to the wind data in Appendix 3. The sites utilised for analysis were Sites 2, 4, 5, 7, 8 and 11 (Figure 2.9) where detailed surveys of beach profiles were obtained. With the presence of berms and other features along the profiles causing an undulating surface, the angle of the potential swash zone for a lake level of 532 msl was used for calculating the $R_{2\%}$ heights. Therefore, these angles may differ to those mentioned in Section 1.4.2 as they represent the average angle of the entire foreshore. The daily maximum H_s and T values obtained by the LAKEWAVE model were used for determining the $R_{2\%}$ heights. A ΔT_{AW} value of 2.8ºC was applied to all the data as it was evaluated while wave recording on the 15/01/11, so it is assumed to be representative of the entire month.

Appendix 5 presents all the results for each of the six specified sites regarding daily run up levels ($R_{2\%}$) over the course of January 2011. Of the six sites, Site 8 is exposed to the longest fetch and receives the largest amount of wave energy. Figure 5.1b graphs the mean daily lake levels for January 2011 at Site 8A and the addition of associated $R_{2\%}$ values to show the total water elevation. Wave recording commenced on the 15/01/11 at Site 8 and the runup results are implemented on this date as opposed to its hindcasted values.

At Site 8 during January 2011 the elevation of the water level frequently surpassed the 532 m mark daily, 77% of the time. The runup heights exceeded the base of the cliff a total of 7 days. The maximum heights above the base of the cliff are misrepresented in Figure 5.1b because it is only the foreshore slope (β), not the near vertical cliff face, taken into account for Equation 5.1. Wave runup surpassing the base of cliff will proceed as water spray directed upwards along the cliff face. Thus, it is expected that waves will act on a higher portion of the backshore than predicted in this analysis.

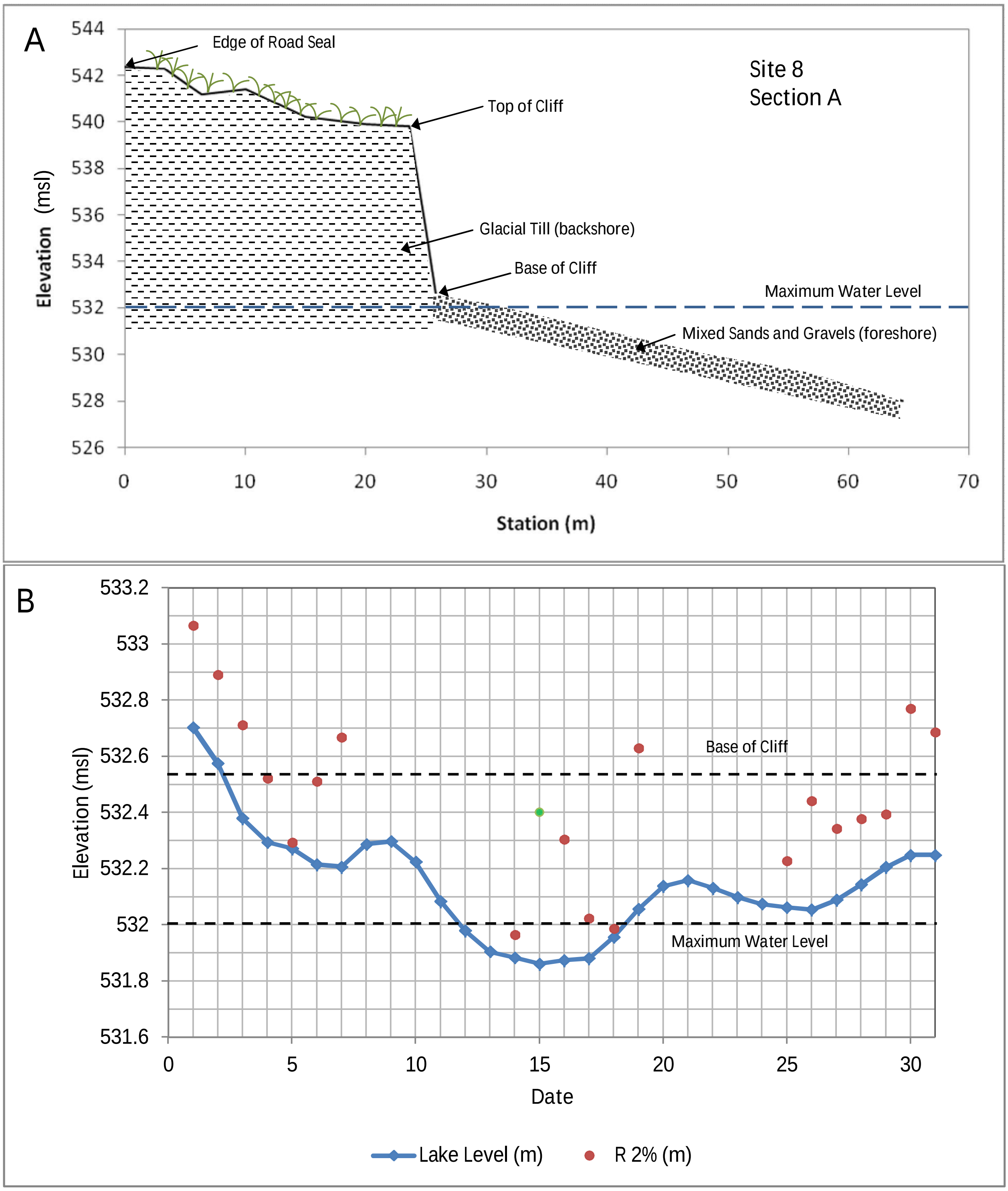


Figure 5.1: A) Beach profile at Site 8A, located on the southern shoreline, elevation exaggerated two times. B) Lake level and $R_{2\%}$ daily maximum values at Site 8 for January 2011. For 11 days no runup values were obtained due to the occurrence of prevailing southerly winds. The green residual highlights a $R_{2\%}$ value based on measured wave data acquired by the XR-620. $R_{2\%}$ is the 2% exceedence of all runups.

Figure 5.2 shows the conditions prior and post a storm event that occurred on 15/01/11 at Site 8. A large proportion of the finer material in the foreshore had been removed after the event, exposing larger boulders. As indicated in Figure 5.1b, the total water elevation did not reach the base of the cliff, which is further evidenced by Figure 5.2. There is no obvious evidence of wave runup eroding the backshore as the presence of driftwood near the toe of the bluff did not seem to move.

To compare the variance of runup elevations at Lake Pukaki Table 5.1 was produced to summarise the maximum $R_{2\%}$ values obtained during a storm event on the 19/01/11. Of all the northerly wind events that happened during January 2011, this event on the 19/01/11 where winds averaged 12.57 ms^{-1} for four hours, was estimated to produce the largest waves. $R_{2\%}$ ranged between 0.35 – 0.83 m, with an average of 0.54 m. Differences in $R_{2\%}$ elevation for Site 7 and Site 11 (0.65 m compared to 0.83 m) reflect the steeper beach slope for the latter site. The beaches on the eastern and western shores are generally narrower and steeper, which leads to an increase in wave runup elevations. Due to the exposure to northerly and southerly winds, sites on the eastern and western shores endure a high frequency of waves acting on the cliffs backing the beaches. This is further highlighted at Site 4B as over the course of January 2011 waves exceeded the base of the cliff for 16 days (52% of the measurement period). In contrast, the backshore at Site 7 was never reached by waves during January 2011 as it consists of a wide and generally flat beach profile.

Unfortunately, wave runup heights were not quantified in the field via geomorphic indicators, such as berm heights, to validate the accurateness of Equation 3.5 utilising a *C* value of 0.9. Lake Hawea's origin and geomorphology is very similar to that of Lake Pukaki's so it is assumed that a *C* value of 0.9 is appropriate. As the foreshore becomes more saturated the *C* value will increase over time as it represents a less porous slope. Therefore, it is anticipated that the wave runup will also increase.

Figure 5.2: A) Site 8 on the southern shoreline on 15/01/11 at 12:15, looking west. B) Post storm on 16/01/11 at 10:30. Note the higher amount of coarser material exposed on the foreshore and the increase in lake level elevation due to wave setup. The driftwood, (red arrow) at the top of the slope doesn't seem to move following the event, indicating wave runup has not reached the base of the cliff.

Table 5.1: Extreme wave runup statistics for a range of sites around Lake Pukaki during a northerly storm event on the 19/01/11. H_s is the significant wave height, T is the wave period, β is the beach slope and $R_{2\%}$ is the 2% exceedance of all runups.

Location	H_s	T	Section	β (°)	$R_{2\%}$ (m)	$R_{2\%}$ + RL 532.06 (m)
Site 2	0.59	2.8	2A	11.3	0.48	532.54
			2B	8.3	0.35	532.41
Site 4	0.7	3.1	4A	9.1	0.46	532.52
			4B	9.6	0.49	532.55
Site 5	0.72	3.2	5A	7.2	0.38	532.44
			5B	7.4	0.39	532.45
Site 7	0.95	3.7	7A	8.9	0.63	532.69
			7B	9.2	0.65	532.71
Site 8	1.09	4	8A	7.0	0.57	532.63
			8B	8.3	0.68	532.74
Site 11	0.84	3.4	11A	13.6	0.83	532.89
			11B	10.2	0.62	532.68

5.3 Wave Power Estimates (*P*)

This section utilises wave power (*P*) to examine the incidence and frequency of storm conditions occurring at Lake Pukaki. Wave power (*P*) ($Nms^{-1}m^{-1}$), also known as wave energy flux , is the rate at which energy is transmitted in the direction of wave propagation across a vertical plane perpendicular to the direction of wave advance and extending down the entire depth of the water column (Demirbilek and Vincent, 2002). It is determined by:

$$P = EC_g \qquad \text{Equation 5.2}$$

Where E is the deepwater energy density or specific energy (Ns^{-1}). It is the total average wave energy per unit surface area, derived by:

$$E = \frac{\rho g H^2}{8} \qquad \text{Equation 5.3}$$

Where C_g is the deepwater wave group velocity (ms^{-1}), given by:

$$C_g = 0.5\,\frac{gT}{2\pi} \qquad \text{Equation 5.4}$$

Wave power (P) is a useful parameter for specifying the wave regime for a lacustrine setting as it takes into account the wave shape, considering both the wave height and wave length. Thus, it deals well with steep waves that are characteristic of South Island alpine lakes.

LAKEWAVE predicted wave data were used to estimate average daily wave power at Site 8, which is exposed to the longest fetch during December 2010 and January 2011. Both December and January have been identified as exhibiting a high percentage of strong northerly wind events. Figure 5.3 graphs the wave power over the specified period as well as the lake level elevation. As previously discussed, the incidence of storm events that occur with high elevated lake levels result in severe erosion of the upper foreshore and backshore. When referring to Figure 5.3 it is interesting to take note of the magnitude and duration of storm activity at Lake Pukaki. In total twenty days exceed 500 $Nms^{-1}m^{-1}$ of wave crest, which ranged from 546 to 2727 $Nms^{-1}m^{-1}$ of wave crest. A cluster of four successive storms that occurred from 23/12/10 to 26/10/12, covering a rise of 0.8 m in water level, all exceeded 1500 $Nms^{-1}m^{-1}$ of wave crest. During this time it is expected that a high degree of widespread erosion has occurred at subsequent elevations along the shore profile with rising lake levels. Another significant storm endured for five days between 30/12/10 and 3/01/11. When compared with the other residuals in Figure 5.3 the affiliated wave power can be considered minuscule. However, it is the incidence of this episode that coincides with when lake levels were at their highest elevation of 532.38 to 532.87 msl. Wave energy is concentrated at the upper limits of the beach profile throughout this event, leading to extensive erosion at the base of cliffs along the backshore.

Allan (1998) applied a similar approach for quantifying storm events for the period June 1992 to May 1996 at Lake Dunstan. From his results, only six days over the four year period exceeded wave power equivalent to 500 $Nms^{-1}m^{-1}$ of wave crest. It was discussed earlier in Section 3.4.1 that in comparison with Lake Dunstan, Lake Pukaki experiences larger wave heights and periods, which has lead to higher wave energy and power estimates. Overall, December has a greater occurrence of high magnitude events, whereas January experiences higher elevated lake levels. Gaps in the data highlights when southerly winds prevailed, which are not intercepted at Site 8 as it is located at the southern end of Lake Pukaki.

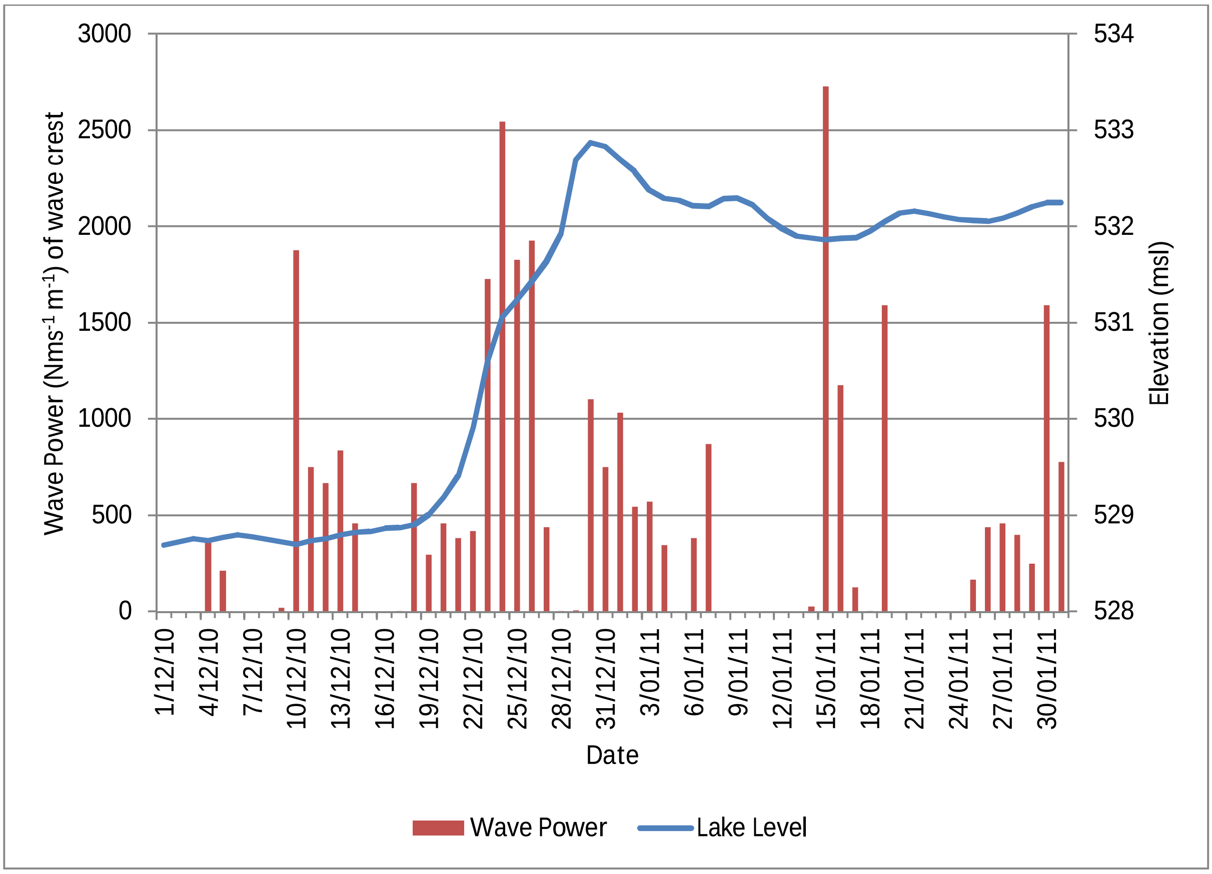


Figure 5.3: Average daily estimates of wave power using hindcasted wave data for Site 8 during December 2010 and January 2011.

<u>5.4 Longshore Sediment Transport</u> (LEXSED formula)

Introduced earlier in Section 2.2.2, longshore currents are a major contributor in terms of sediment transport on alpine lakes. Longshore sediment transport rates have been estimated during the month of January 2011 for the same six sites (Sites 2, 4, 5, 7, 8 and 11) assessed in Section 5.2.1, where the wave runup ($R_{2\%}$) was evaluated. It was noted that on January 2011 lake levels were at the upper end of the operating range and there was a high frequency of strong northerly winds.

For his research at Lake Coleridge, Dawe (2006) developed an equation that estimates longshore sediment transport rates on mixed sand and gravel beaches. He found that longshore transport was a function of wave height, wave period, beach slope, swash velocity and the wave approach angle. The LEXSED formula, which stands for low energy mixed sediment transport, produces figures in cubic metres per second ($m^3\ s^{-1}$) and is a total integrated rate for the whole swash zone.

$$Q_1 = k \left(\frac{H_s{}^3}{L_o}\right) V_{sw}\ \sin \alpha_b \qquad \text{Equation 5.5}$$

Where k is a coefficient, α_b is the breaker angle (radians) and V_{sw} is the swash velocity (ms^{-1}), which is given by:

$$V_{sw} = k \sqrt{\frac{H_s/T_z}{\tan \beta}} \qquad \text{Equation 5.6}$$

Where k is also a coefficient. The coefficients are ascertained by field investigations and as a result are site specific. They take into account variables such as beach porosity, which influences wave energy absorption. At Lake Coleridge Dawe (2006) determined k = 0.02 for Equation 5.5 and k = 0.27 for Equation 5.6. Dawe (2006) states that the LEXSED formula is suitable for other South Island alpine lakes, in low energy (H < 1.0 m), coarse grained beaches.

Estimated longshore sediment transport rates per hour over the month of January 2011 for Sites 2, 4, 5, 7, 8 and 11 (Figure 2.9) are presented in Appendix 5 and Table 5.2. The gross and net rates are also included in Appendix 5 and Table 5.2. Both the k values obtained by Dawe (2006) have been applied for the purposes of this study. Dawe (2006) used the wave height parameter H_{rms} instead of H_s for Equations 5.5 and 5.6, however he did state that using H_s would also be adequate. Only the strongest wind events for each day during January 2011 were used for estimating wave statistics in LAKEWAVE, which were then implemented in the longshore transport formulas. In all his sampling, Dawe (2006) could only measure acceptable rates of longshore transport in the severest conditions. The maximum predicted H_s over the course of January 2011 was 1.09 m at Site 8, as shown in Table 5.1, which agrees with the wave height limitations of the LEXSED formula. Predicted wave heights (H_s) <0.1 m were not considered for this analysis. The average beach slope (β) was taken for each of the six studied sites. α_b values were determined by LAKEWAVE according to the wind direction, thus the wave propagation direction, relative to the shore orientation. Simply multiplying the result of Equation 5.5 with a value of 3600, converts the rate to cubic metres per hour ($m^3\ h^{-1}$).

Table 5.2: Summary of longshore sediment transport rates for several sites at Lake Pukaki during January 2011. LST is longshore sediment transport.

Location	LST rate ($m^3\ h^{-1}$)			LST rate (m^3) for January 2011	
	Mean	Minimum	Maximum	Gross	Net and Direction
Site 2	0.088	4.07×10^{-3}	0.295	8.575	8.346 *South*
Site 4	0.151	5.76×10^{-3}	0.510	15.420	15.125 *South*
Site 5	0.165	6.59×10^{-3}	0.562	16.910	16.551 *South*
Site 7	0.066	3.21×10^{-3}	0.182	5.817	5.817 *South*
Site 8	0.083	4.45×10^{-4}	0.249	6.881	6.880 *East*
Site 11	0.197	7.42×10^{-4}	0.748	22.112	21.890 *South*
Overall	0.129	4.45×10^{-4}	0.748	-	-
Dawe (2006) Lake Coleridge	0.074	1.10×10^{-5}	1.154	-	-

The eastern and western shores exhibit the largest amounts of longshore sediment transport due to the bi-directional wind regime at Lake Pukaki. The main transport direction is southwards because of the strong frequent northerly winds. According to the LEXSED formula the longshore sediment transport rate at Site 8 is predominantly eastwards, which does not agree with what was observed in the field. A groyne situated immediately west of Site 8 showed a build up of sediment on the eastern side, which indicates that longshore transport is moving primarily westwards. The nearshore bathymetry at Site 8 may exert an influence on the wave approach angle, which cannot be taken into account from predictions.

Measured hourly rates acquired by Dawe (2006) at Lake Coleridge are included in Table 5.2. On average, through a 1 m cross section of the beach, it is predicted that Lake Pukaki experiences higher longshore transport rates. However, Lake Coleridge obtained the highest maximum hourly rate of 1.154 $m^3\ h^{-1}$. It should be noted that Dawe (2006) quantified longshore transport over the course of a year, whereas predictions for this study only cover one month. The frequency of extreme events is higher when examined over a longer period.

The reliability of using the LEXSED formula for estimating longshore transport rates at Lake Pukaki is subjective. The analysis assumes that the Lake Coleridge nearshore process environment and beach morphology is very similar to that of Lake Pukaki. It has already been pointed out, in Section 3.4.1, that Lake Pukaki experiences a greater wave energy regime than Lake Coleridge. Comparison of the shoreline morphology and sediment characteristics of the two lakes is difficult, as this was only studied in minor detail at Lake Pukaki. On-site measured longshore transport rates are necessary to evaluate applicable k values, used for Equation 5.5 and 5.6, which represent the site conditions of Lake Pukaki. In hindsight, the LEXSED formula does however illustrate that longshore currents are a significant process for sediment transport at Lake Pukaki. It also backs up previous assumptions that the eastern and western shores are subject to large rates of longshore sediment transport in comparison to the southern shore.

5.5 Conclusions

The LAKEWAVE model has been used to describe the wave environment about the Lake Pukaki shoreline in terms of its maximum energy potential. The lake level elevation has been identified as a controlling factor for the level at which wave action is concentrated on the beach profile. It is storm conditions coincident with high lake levels that can lead to significant erosion along the backshore. Extensive erosion at the base of cliffs can result in eventual slope instability and failure.

Calculations of extreme daily runup elevations have been estimated for several sites at Lake Pukaki during January 2011. The strongest wind event within this timeframe was on the 19/01/11, which produced winds of 12.57 ms^{-1} from the north. Estimates of $R_{2\%}$ for this event ranged between 0.35 – 0.83 m, with an average of 0.54 m. Total water levels were assessed, which are the combination of mean lake levels with the addition of calculated runup of storm waves. During high lake levels, the width of the foreshore is shortened and becomes very narrow, especially on steeper beaches along the eastern and western shoreline. This has lead to an increase of runup elevation for the beaches located on the eastern and western margins of Lake Pukaki. At Site 4B wave runup intercepted the base of the cliff for 16 days during January 2011. When wave runup exceeds the base of the cliff the resultant swash is directed vertically up the cliff face.

As there is a high incidence of storm activity during the summer months, average daily wave power estimates were calculated during December 2010 and January 2011 for Site 8, which is exposed to the longest fetch. In total twenty days surpassed 500 $Nms^{-1}m^{-1}$ of wave crest and ranged from 546 to 2727 $Nms^{-1}m^{-1}$ of wave crest. December 2010 was shown to have a higher occurrence of storm events than January 2011. However, it is when lake levels are at their maximum elevation that wave energy is concentrated at the upper limits of the beach profile. For this reason more extensive erosion along the backshore has likely to have occurred during January 2011.

Longshore transport rates were estimated using the LEXSED equation, formulated by Dawe (2006), for a range of sites at Lake Pukaki during January 2011. Predicted Q_l rates were on average higher at Lake Pukaki than what was measured at Lake Coleridge by Dawe (2006). The eastern and western shorelines are subject to a larger rate of sediment transport under longshore currents compared to the southern shoreline. It is these strong longshore currents during high lake levels that lead to a loss of natural beach armouring immediately in front of the eroding cliffs.

The wave runup, wave power and longshore transport estimates discussed here are analysed in further detail in Chapter 7 in relation to the performance of the rock revetments currently in operation at Lake Pukaki, which are introduced and discussed in the next chapter.

Chapter 6

LAKESHORE PROTECTION AND PERFORMANCE

6.1 Introduction

In an effort to protect the road network bordering the Lake Pukaki shoreline, a number of different erosion control techniques have been put in place to combat sustained shore retreat. This chapter focuses on the functionality and performance of shore erosion control structures currently in operation at Lake Pukaki.

It is important that the existing shore protection structures do actually function as protection, and do not adversely affect the lakeshore environment (James *et al*. 2002). Overall, there has been a lack of quantitative assessment of shore protection structures in terms of their performance, and little is usually known in regard to the nearshore process regime. An invaluable tool in the implementation of such structures is a sound understanding of the active nearshore processes based upon investigations, measurement and monitoring. The assessment of structure performance for this study is achieved through the identification and consideration of the contributing processes causing erosion, including winds, waves, currents and lake levels, that have been measured and analysed in Chapters 3, 4 and 5.

There are many types of coastal protection options, each suitable for a definite purpose and environment, these methods are discussed further in Section 6.2. The history of shoreline management and the implementation of erosion control structures at Lake Pukaki is summarised in Section 6.3. The funding of coastal defence schemes relies on an economic assessment of whether the benefits of defence outweigh the cost of construction (Reeve *et al.* 2004). The cost of construction depends on the types of coastal protection chosen and materials used, which is examined in Section 6.4. The performance of revetment structures currently in use at Lake Pukaki is analysed, in terms of field observations and theoretical stability, in Section 6.5.

6.2 Available Options for Erosion Control

The term 'coastal protection' is generally used to descibe schemes designed to protect an existing shoreline from further erosion (Reeve *et al.* 2004). This term not only applies to open-coast settings but also to lacustrine environments. Various measures can be used for coastal protection: indirect measures take away the causes of the problem whereas direct measures prevent or mitigate the immediate effects of the problems (Pilarcyzk, 1990). This study is concerned with direct measures used for lakeshore protection; which can be divided into two categories. The first is referred to as 'hard engineering' where structures are constructed on the shoreline to resist the energy of nearshore processes. Such structures include groynes, breakwaters and seawalls. The second direct measure is referred to as 'soft engineering', which aims to work in sympathy with the nearshore processes by mimicking natural defense mechanisms. For example, beach renourishment is a soft engineering method. Elements of hard and soft engineering are often used together to provide an optimal shoreline defense scheme.

As stated by Kirk (1988, p 150):

> *"Whatever strategy is adopted, it is important to realise that shoreline management is an ongoing commitment. There are no 'one-shot' solutions to shoreline erosion, and no maintenance-free structures. There are thus only choices to be made between various mixes of capital and maintenance costs and between the broader costs and benefits of a range of strategies."*

The advantages and design principles concerning offshore breakwaters are not discussed in the following sections. Their implementation and operation is not considered a feasible option for erosion control at Lake Pukaki as they would entail high costs during construction due to the steep nearshore shelf.

6.2.1 Seawalls

Seawalls are the most widely used option for shoreline defence. The term ‘seawall’ also applies to revetments, dykes, gabion baskets, bulkheads and sills (Silvester and Hsu, 1997). The primary functions of a seawall for coastal protection are to protect the shoreline from the effects of erosion; and to protect land and property from damage by wave energy. It should be noted that it is only the land behind the wall that is protected and not the beach in general. On a naturally eroding shoreline the introduction of a seawall to prevent further landward regression will not stop the overall erosion process, and may in certain instances aggravate the situation (Reeve *et al.* 2004). A new wall may cut off the natural supply of material to the littoral regime, and in this way the shoreline erosion is replaced by erosion of the foreshore immediately in front of the seawall.

Modern design practice would normally dictate that any new structure should have as gentle a slope as possible and be protected by a cover layer that dissipates as much wave energy as possible (Reeve *et al.* 2004). However, this is not always the case due to the availability of space along the beachfront. It is preferred that the wall be located as far landward as possible to minimise both interference with the coastal regime and the hydraulic loads.

Each seawall type consists of three elements: body, including front face and core; toe; and crest. The body is the major part of the sea wall and has a great influence on its performance. The front face of the seawall determines the classification used for the body. For the various types of seawall it is convenient to classify them as: sloping (<45º); vertical (>45º); porous, where the face is permeable to wave action (e.g. riprap); and non-porous, where the face is not permeable (e.g. concrete wall) (Thomas and Hall, 1992). At Lake Pukaki a rock revetment (sloping, porous) is backed by a gabion wall (vertical, porous) at Site 9, shown in Figure 6.1. A reno mattress beneath the rock revetment restricts any potential undermining of the toe. Toe erosion is the most common cause of seawall failure, which emphasises the importance of toe design. Toe stability is essential since failure of the toe will generally lead to overall failure of the wall. The toe needs to extend down to a level sufficiently below the lowest likely beach level or be founded in strata that is relatively resistant to erosion. Introducing a rock or concrete apron into the toe construction has the advantage of taking the scour problems further away from the foundation of the wall. The primary function of a crest is to prevent overtopping (Thomas and Hall, 1992), and so the crest height relative to the highest water level and wave run-up is important.

Figure 6.1: Rock revetment backed by gabion wall at Site 9, facing southwest. Each tier of the gabion wall is ~0.9 – 1 m high. The rock revetment is founded on a reno mattress which prevents any possible undermining of the structure. Photo taken on 18/11/10.

6.2.2 Groynes

The objective of constructing a groyne or groyne system is to stabilise a stretch of beach against erosion, where that erosion is primarily due to a net alongshore loss of sediment. Groynes can serve to protect an area or to maintain a wide recreational beach by trapping longshore drift or to reduce rates of longshore transport during severe storm events. However, without an adequate supply of either natural or artificial beach material, groynes are not effective in raising beach levels (Thomas and Hall, 1992). They are narrow structures, often of rubble-mound or sheet-pile construction, that are usually built perpendicular to the shoreline. The rubble-mound structures are most common, especially in high energy environments, consisting of armourstone or concrete armour units. An example of a rubble-mound rock groyne at Lake Pukaki is shown in Figure 6.2.

Figure 6.2: Groyne at Site 9. Longshore currents predominantly moving westwards as there is noticeably more material deposition on the eastern side. Photo acquired on 18/11/10 from the same position as the photo taken in Figure 6.1, except facing north.

The functional design of groynes requires a knowledge of the sediment budget and longhore material transport processes at the site. Groynes might be considered if the amount of sediment leaving the project area by longshore transport exceeds the amount entering. Most longshore transport takes place in the surf zone between the outermost breaking waves and the shoreline; and on the beach face below the limit of wave runup. Accordingly, groyne length from the shoreline should be established based on the surf zone width. To avoid outflanking, the landward end of a groyne should be connected to a non-erodible shore parallel defense such as a cliff, seawall or revetment (Reeve *et al.* 2004).

6.2.3 Beach Nourishment

Beach nourishment has been categorised as a soft option for beach stabilisation schemes. It is the addition of sediment to replenish an eroding part of the shoreline. It is often the most satisfactory means of protecting a shoreline as it provides the necessary reservoir of material that allows a beach to respond relatively normally to differing nearshore processes (Reeve *et al.* 2004). It entails finding a suitable source of material that is compatible with, but not necessarily identical to the material that occurs on the beach.

Bad judgements are usually forgiven more easily in the design of a beach renourishment scheme than for other types of coastal protection. The needed volume is directly proportional to the annual amount of losses out of the stretch of shore under consideration (Van de Graaff *et al.* 1991). If the initial estimated annual losses are incorrect, only the estimated lifetime of the nourishment has to be adjusted and an earlier/later repetition will be necessary. Beach nourishment programs are considered a very flexible means for coastal protection. The rate of application depends on the energy state of the beach under consideration. The expected lifetime between applications can shorten due to the occurrence of storms, as a beach experiences the biggest sediment losses during storm events. The existence of nearby groyne structures can aid in the effectiveness of beach nourishment programs as the amount of sediment lost from the beach cell is restricted.

The main disadvantage is that nourishments have to be repeated from time to time (Van de Graaff *et al.* 1991). The cause of the erosion problem is not solved, as erosion continues along the shoreline. However the costs of beach nourishment are inexpensive compared to the cost of other coastal protection measures, including maintenance costs required for hard engineered structures, such as groynes or revetments.

6.2.4 Relocation

Planning and relocation of assets at risk is a far better preventative measure against shoreline erosion than an engineered landscape. It is regarded as a one-time cost option, permanently removing assets away from the zone at risk to shoreline erosion (Kirk, 1988). Managed retreat is simply a mechanism for putting a value on the beach over assets bordering it, which involves relocating property or infrastructure further back behind the backshore. This allows the beach to function in a natural way by being able to adapt its planform shape and cross section in different conditions, such as storm events. Thus, allowing the beach to develop an equilibrium profile.

Due to economic reasons it is not always feasible to govern the relocation option. A cost-benefit analysis is often required against other mitigation techniques to verify if relocation is a viable choice.

6.3 History of Shoreline Erosion and Mitigation at Lake Pukaki

Following successive raisings of the level of Lake Pukaki and the frequent coincidence of high lake levels and storm activity, the roads in close proximity to the lakeshore have become under threat due to extensive erosion. These roads include State Highway 80 (SH80) on the western shore, State Highway 8 (SH8) along the southern shore and Braemar and Hayman Road on the eastern shore. Opus (2000, 2009) has recognised several hazardous sites, originally identified by the Ministry of Works in 1988, where erosion is prominent and there is a potential risk to the road network. Eleven of the field sites of this present study are based on the location of the hazard sites identified by Opus (2000, 2009).

In an effort to reduce the rate of retreat of the shoreline, erosion control structures have been constructed along the shores of Lake Pukaki since 1987 to early 1988. These works display various degrees of success and as part of their design require ongoing maintenance. It is assumed that the maintenance recurrence interval of the shore protection structures is likely to reduce over time as they become better designed. The construction and maintenance of shore protection structures are carried out during periods of low lake levels, when most of the foreshore is exposed. The remedial works undertaken to date have comprised gabion walls, rock revetments, reno mattresses, groynes, beach nourishment and steel rail and rope retaining fences. The history concerning the instalment of shore protection measures at Lake Pukaki, which has been summarised from Opus's (2000) "Lake Pukaki - Five Year Plan for Shoreline Management 2000 – 2005" and Opus's (2009) "Lake Pukaki Shoreline Erosion Inspection, May 2009", is presented in Table 6.1.

Table 6.1: The present study field sites in relation to the hazard sites identified by the Ministry of Works and the history of shoreline erosion mitigation at each site. RR is rock revetment. See Figure 2.9 for location of these.

Present Studies Field Sites	Ministry of Works / Opus Hazard Sites	Shore Protection Measures
Site 1	Site 2	*Autumn 1995* – RR constructed, covering 100 m stretch of the backshore. *Nov 1995* – Height of RR raised several metres above maximum lake level. *Dec 1999* - RR extended due to flanking, now covering 300 m stretch. Original RR material topped up and repacked. *June 2008* – Existing protection augmented, especially along southern stretch.
Site 2	Site 4	*Mid 1988* – Gabion wall built. Soon after construction it failed due to structural instability and foundation undermining. *Dec 1989* – Extensive reconstruction and protection works undertaken, including instalment of RR and a steel rail and rope retaining fence. *Dec 1991* – Subsided sections refilled with rock. *Oct 1995* – Slumped upper batter areas repacked. Riprap lake side of retaining fence repacked and reshaped to original profile. *March 2001* – Further repairs due to storm damage in Feb 2001 *May 2008* – Eroded/slumped sections replenished with rock and tied into existing batter slope.
Site 3	Site 5	*Mid 1988* – 120 m of road reinstated and protected. Rail and wire rope fence constructed to retain fill material. Following year wire mesh between rails failed. *Spring 1996* – Fillet of coarse gravel from Landslip Creek was placed along toe of the embankment.
Site 4	Site 6	*Mid 1988* – 120 m of embankment reinstated and RR constructed. *Spring 1996* – Fillet of coarse gravel from Landslip Creek placed along toe of embankment. *June 2003* – Scattered rock retrieved and repacked. *Nov 2005* – Isolated pocket of erosion filled and slope rearmoured. *May 2008* –Existing protection was augmented where slumping occurred or riprap was scarce.

Site 6	Site 8	Rock from early protection works, ~35 years ago, litters the beach. *May 2008* – Scarp filled with gravel bund. RR constructed at toe of cliff.
Site 7	Site 8A	Existing rock protection work placed many years ago (exact time unknown). *Nov 2005* – RR extended 40 m to the south.
Site 8	Site 9	*Oct 2001* - Two rock groynes and 70 m of RR constructed. Toe excavated during placement to limit risk of undermining. *Nov 2005* – RR extended 30 m east and west.
Site 9	Site 10	*Feb 1988* – 160 m long gabion wall and 6 m wide reno mattress constructed. Mattress was later extended at both ends. *Feb 1989* –Mattress with abrasion damage overlaid with terramesh (PVC coated wire) panels. *Feb 1992* – Sections of mattress repacked. Coarse gravels placed a toe to prevent undermining of mattress. *May 1995* – Coarse gravel placed at eastern end to prevent further slumping of mattress. *Feb 1996* – Rock apron at toe of mattress was augmented. Damaged mattress sections replaced. 2 trial rock groynes constructed at the middle and west end of the gabion wall. *May 1998* – Further repacking of 25 reno mattress sections. *Dec 1998* – RR built on top of entire reno mattress.
Site 10	Site 12	No mitigation measures taken, except tree removal.
Site 11	Site 15	RR documented to have been replenished in the late 1980's. *Sept 2001* – Slumped section adjacent to bridge refilled with coarse rock. *Early 2007* – Riprap placed at site of embankment slip.
Site 12	Site 16	*Dec 1987* – Gabion wall and reno mattress constructed. *Dec 1989* – Mattress holes patched and refilled. Beach nourishment gravels placed on top of mattress, up to 0.5 m in diameter, sourced from Twins Stream. *Dec 1993* – Beach nourishment gravels extended 160 m north. *May 2008* – Major repairs undertaken. Concrete facing was poured at the toe of the gabion wall along the top section of the mattress. Damaged mattress sections overlaid with terramesh panels, followed by a layer of nourishment gravels placed over the entire mattress. Gravel within eroded pockets of cliff face to the north was augmented.

Since 1988, in co-occurrence with the identified hazardous sites and implementation of protection structures, shoreline monitoring and reporting has been carried out on a bi-annual basis or following specific storm events (for example Opus, 2000, 2009; Single, 2006; Single and Bunting, 2003). Not only has the extent of shore erosion been evaluated during each inspection but also the general integrity and performance of the protection structures in operation has been assessed.

Surveys have also been completed at times in conjunction with inspections for examining profile changes. Meechia (1976) initially set up a shoreline survey network prior to the raising of the lake in 1976. Most of the profiles established by Meechia were resurveyed and new survey sites were established in October 2003 by Elliot Sinclair & Partners Limited. A total of forty-two relocatable sites are installed along the Lake Pukaki shoreline, which are continually resurveyed to monitor shoreline change.

Revetments are by far the most common strategy for mitigating wave-induced erosion at Lake Pukaki. All of the sites discussed in Table 6.1 with rock revetment structures are analysed in terms of their performance in Section 6.5, except for Site 6. It should be noted that the extensive revetment structures near the information centre in the southeast corner of Lake Pukaki are not considered as one of the hazardous sites outlined by Opus (2000, 2009), as this is part of the dam structure. Along this stretch of the shore, the riprap is over-designed, consisting of very large angular boulders, used to protect the dam and the canal intake.

6.4 Material Properties and Availability

The extensive moraine deposits stretching beyond the southern and eastern shoreline have provided an abundance of boulder sized material available to be utilised as riprap for shore protection structures at Lake Pukaki. Boulders are frequently exposed at the surface of the moraine, which is typically farmland, or are within 2 – 3 m depth. Greywacke boulders are the most common, which are an indurated sandstone of the Torlesse Terrane. They are typically unweathered, have an unconfined compressive strength (UCS) in the range of 100 to 200 MPa, a dry density of ~ 2600 kg/m^3 and perform well in LA abrasion and sodium sulphate tests (URS, 2004). The quarried greywacke blocks have a fairly wide size distribution and are mainly angular to sub-angular, which is typical of riprap (Hughes, 2001). They possess good interlocking properties for use as rock armour on revetments due to their irregular shape. Aside from the greywacke, a small proportion of schist boulders are also found within the moraine deposits. Riprap material is also sourced and excavated from nearby stream beds and alluvial fans that flow into Lake Pukaki. The associated greywacke boulders are typically sub-rounded to sub-angular in shape.

Exploiting material from a source in close proximity to the project site is a cost-effective solution, in terms of extraction, transportation costs and construction. Another advantage of using local materials is that of long term availability, which is necessary for maintenance (Thomas and Hall, 1992). As stated earlier by Kirk (1988), there is no 'one-shot' solution for erosion control and structures are required to be regularly maintained. In an effort to quantify the amount of material available, required for the construction and maintenance of shoreline structures, investigations and evaluation of rock armour sources in the vicinity of Lake Pukaki have been undertaken by engineering consultancies, including Opus International Consultants Ltd and URS New Zealand Ltd. Information on the yield of potential quarries, including the maximum size and size distribution of rock armour is essential for continued management.

An alternative option to rock boulders, for shore protection purposes at Lake Pukaki, would be concrete. For coastal structures, concrete can be mixed on-site and placed in-situ to construct seawalls. Impermeable concrete seawalls are an uncommon feature of the shore protection network at Lake Pukaki due to their inability to dissipate wave energy, generally leading to accelerated erosion rates lakeward of the structure. Concrete also can be manufactured off-site to produce pre-cast armour units, used to construct revetments, groynes and breakwaters. They are developed to provide a high degree of interlock and hence stability, while at the same time being robust enough to withstand breakage (Reeve *et al.* 2004). The transportation of the pre-cast blocks and the careful consideration required for placement of the individual units during construction can lead to these structures being a very expensive option. They are generally utilised when armourstone of sufficient size cannot be obtained in the required quantities and therefore are not necessary for the Lake Pukaki setting as there are high amounts of available armourstone in close proximity to the site. However, on occasion random concrete blocks were found amongst the revetments, especially at sites along the eastern shore (an example is shown in Figure 6.3). It seems they were not initially fabricated for coastal protection purposes but are instead waste material derived from another project.

Figure 6.3: Concrete block (~0.95 m) found within a revetment of predominantly greywacke boulders at Site 4. Photo taken on 6/11/10.

6.5 Revetment Performance Assessment

A similar approach undertaken by Lefebvre *et al.* (1992, 1995) was carried out for this study for assessing riprap stability of rock revetments. Lefebvre *et al.* (1992, 1995) conducted a detailed field investigation on fourteen riprap sites across the La Grande hydro-electric complex in northern Quebec, Canada. Initially sites were visually examined in regards to the general riprap state and if any signs of failure were evident. The in-situ gradation of sites were then assessed so that a back analysis could be completed, whereby the theoretical stability of the riprap was compared to the maximum wave heights experienced since the structures have been in operation. The methodology and results concerning the field performance of rock revetments in place at Lake Pukaki via visual observations and in-situ gradation are presented in Sections 6.5.1 and 6.5.2 respectively.

It should be noted that only the revetments at Sites 1, 2, 3, 4, 7, 8, 9, and 11 (as located in Figure 2.9) were assessed in terms of their structural performance. Other nearby shore protection structures that aid in the stability of the revetments at these identified sites are also discussed.

6.5.1 Visual Observations (Six Rules)

Methodology

The rock revetments on Lake Pukaki have been assessed through visual observations according to the 'six rules' that are used by the US Army Corps of Engineering Research (CERC, 1984b). These six rules must be satisfied in design and construction of shore protection works in order to mitigate lakeshore erosion. The rules are a useful guide for assessing the performance capabilities of the structure under the proposed water level operating regime (Single and Kirk, 2003). The rules are as follows:

1.) Provide adequate protection for the toe of the structure so that it will not be undermined;
2.) Secure both ends for the shore protection works against flanking, to prevent the acceleration of erosion of the unprotected shore at the ends of the structure;
3.) The foundation of the structure must be competent to withstand excessive settling;
4.) Use material that is heavy and dense enough that waves will not move individual portions of the protection;
5.) The structure must be of a height that it cannot be overtopped (spray overtopping is acceptable, but not 'green' water); and
6.) All voids between individual pieces of protection must be small enough as to not allow the underlying material to be washed out by wave action or by ground water seepage.

A 'pass' or 'fail' ranking was awarded for the general integrity of the revetment in regards to each of the six rules. The visual assessments for each site are summarised in Table 6.2. According to Burcharth and Hughes (2003), failure is damage that results in structure performance and functionality below the minimum anticipated by design. However, there are no stated minimum design criteria for these structures. Failure was assessed depending on the severity of damage, if any, and the cause of damage in relation to six design rules.

Table 6.2: Revetment examination according to the 'Six Rules' for shore protection structures design and performance. RR is rock revetment.

Site	Rule #1 Toe	Rule #2 Flanking	Rule #3 Foundation	Rule #4 Weight	Rule #5 Height	Rule #6 Void Space
1	*Pass* No undermining of toe.	*Pass* None evident.	*Pass* No settling noticed.	*Pass* Well interlocked, large boulders.	*Pass* Sparse vegetation on backshore, no obvious signs of overtopping.	*Pass* Small voids as boulders are well interlocked.
2	*Pass* No undermining of toe.	*Fail* Evidence of scouring along southern edge.	*Pass* Few pockets of settling near top of RR, not that significant when compared to the size of the RR.	*Pass* Well interlocked, large boulders. Rail and rope fence adds to stability of RR.	*Pass* Top of RR is at road level, spray may reach road during storm conditions.	*Pass* Small voids as boulders are well interlocked.
3	*Pass* No undermining of toe.	*Pass* None evident.	*Fail* Areas of settling, very loose easily eroded silty backshore, large erosion scarps along cliffs.	*Fail* Small (~0.3 m) boulders rolling down RR. Wire mesh between steel rails has deteriorated.	*Pass* Top of RR ~3 m above lake level of 532.4 msl.	*Pass* Small void spaces due to uniformly sized riprap.
4	*Pass* Minor scour but no slumping seen.	*Pass* No gaps evident behind edges of RR, though severe erosion between sections.	*Pass* No settling noticed.	*Pass* Well interlocked, large boulders, few scattered.	*Pass* RR seems high enough, presence of fines on top due to unstable slope behind RR.	*Fail* Small to coarse gravel exposed in large voids near toe.
7	*Pass* No undermining of toe.	*Fail* Scour behind edge of structure at northern end.	*Pass* No settling noticed.	*Pass* Only some of the smaller boulders are scattered.	*Fail* Erosion along backshore, high amount of fines deposited on top of RR.	*Fail* Large exposed voids in places.
8	*Pass* Toe dug out during construction to avoid undermining.	*Pass* Scouring on western edge but no gap has formed between the RR and backshore.	*Pass* No settling noticed.	*Pass* Well interlocked, get smaller towards the top, none seem to have moved.	*Pass* Vegetated backshore, no fines noticed at top of RR.	*Pass* Small voids as boulders are well interlocked.
9	*Pass* No undermining, aided by presence of reno mattress.	*Pass* None evident.	*Pass* No settling, founded on reno mattress.	*Pass* Well interlocked, very large boulders.	*Pass* Backed by gabion baskets, 3m above RR.	*Pass* Only some voids at toe.
11	*Pass* No undermining of toe.	*Pass* None evident.	*Pass* No settling noticed.	*Pass* Well interlocked, large boulders, few rolled down RR.	*Pass* Vegetated backshore, RR top ~2 m above lake level of 532.4 msl.	*Pass* Small voids, large cobbles, few fines.

Discussion of Results

Abiding to the six rules set by the US Army Corps of Engineering Research, the revetments at Sites 1, 8, 9 and 11 have performed considerably well. In particular, Sites 8 and 9 on the southern shoreline had riprap that consisted of very large sub-angular boulders and showed minimal signs of movement. Both these sites have groyne structures operating in the near vicinity, which have helped to create a buffer of beach sediments. However, a section of severe erosion and scouring ~10 m in length was evident on the western edge of the rock revetment at Site 8, shown in Figure 6.4. Undercutting in the cliff side may eventually lead to failure. No scour has yet occurred behind the structure at its edge, creating a gap between the rock revetment and the backshore.

Figure 6.5 shows the formation of a gap behind the northern edge of the revetment at Site 7. This has developed due to outflanking and has lead to the structure becoming disconnected with the backshore. The gap between the structure and backshore will continue to grow as accelerated rates of erosion is concentrated in this region. This phenomenon was also observed at the southern end of Site 2.

At several revetments, including Sites 4, 7 and 11, some of the smaller riprap boulders have become displaced and rolled down the slope profile. The steepness of the revetment at Site 11, situated on the western shore, is the main reason for rock mobility, as it is one of the steepest revetments at Lake Pukaki (30-31º). The total percentage of scattered boulders at Sites 4, 7 and 11 is not considered to have threatened the structural integrity of the revetments but can lead to large void spaces to form exposing finer material. This was noticed to have occurred at Sites 4 and 7 where the removal of boulders had uncovered prominent areas of underlying beach sediments. This is a major issue for Site 7 as significant erosion has occurred in pockets where there is none to very little protection. Overtopping is also apparent at Site 7 due to erosion observed behind the revetment.

Figure 6.4: Cliff section exposed on the western edge of the rock revetment at Site 8. Prominent undercutting of cliffs and pockets of erosion. Sections at top of cliff showing signs of toppling and failure, delineated by red arrow. Photo taken on 15/01/11.

Figure 6.5: Gap forming behind eastern edge of revetment at Site 7, looking south. Photo taken on 31/08/10.

Figure 6.6: Large section of the riprap at Site 3 has been eroded out due to the unstable silty backshore, looking north. This has lead to the formation of a large scarp to the right of the picture, 1.5 m from the roads edge. Boulders have continued to roll down the face of the revetment. The wire mesh between the steel rail posts, used to retain the material, has disappeared. Photo taken on 5/03/11.

Overall, the performance and functionality of the revetment at Site 3 was quite poor. The riprap at Site 3 has been sourced from Landslip Creek, which is generally sub-rounded in shape and ~0.3 m in diameter. The poor interlocking capability and steep profile has lead to a high proportion of the smaller boulders to roll down the slope, illustrated in Figure 6.6. The steel rail and rope, retaining fence that was installed to combat this has undergone considerable damage and the wire mesh between the rails has almost disappeared. Sections of the revetment have been eroded out due to the instability of the backshore at Site 3, which consists of a high proportion of silt. This has caused several erosion scarps to form along and at the edges of the site within 1.5-2 m from the roads edge. The poor foundation conditions have also lead to excessive settling of the overlying riprap. In comparison, Site 2's revetment only demonstrated localised areas of moderate settlement, shown in Figure 6.7. As opposed to what was observed at Site 3, the structural integrity of the revetment at Site 2 has not been jeopardised.

Figure 6.7: Area of settlement near the top of the revetment at Site 2. These pockets are likely to have formed by a combination of larger boulders being removed and localised settlement. Photo taken on 5/03/11.

6.5.2 Rock Armour Stability Analysis

Site Selection and Procedure

The in-situ gradation was also measured for the same rock revetments that were visually assessed, except for Site 3 as the revetment was too unstable to walk along. Either one or two surface samples were measured at each site within a 4 x 8 m perimeter. These sampled areas are meant to represent the gradation of the entire rock revetment, so if any damaged sections were evident they were implemented in the surface sample. Within each 4 x 8 m surface sample three horizontal scanlines were produced, as shown in Figure 6.8. The three major axes of individual boulders were measured at 0.5 m intervals along each scanline. If the same boulder appeared at consecutive 0.5 m increments another boulder adjacent to the scanline was measured. Rocks less than 10 cm in diameter were not considered. Approximately sixteen boulders were measured along each scanline, with a total of around forty-eight measured for the entire surface sample.

Figure 6.8: Revetment scanline layout at Site 7. The three dimensions of individual armourstone units are measured at 0.5 m intervals along each of the three horizontal scanlines. Photo taken on 17/9/10.

The nominal diameter (D_n) of each block was obtained by:

$$D_n = (a\,b\,c)^{1/3} \qquad \text{Equation 6.1}$$

Where a, b and c are the three major axes. The subsequent mass (M) of each block is then calculated by:

$$M = \rho_r\,{D_n}^3 \qquad \text{Equation 6.2}$$

Where ρ_r is the mass density of rock, which is 2600 kg/m^3. A mass distribution curve was then established for each sample by methods outlined by Latham *et al.* (2002) to obtain M_{50}, which is the median mass of rock grading given by 50% on the mass distribution curve. Figure 6.9 demonstrates how M_{50} was estimated for a sampled section of riprap.

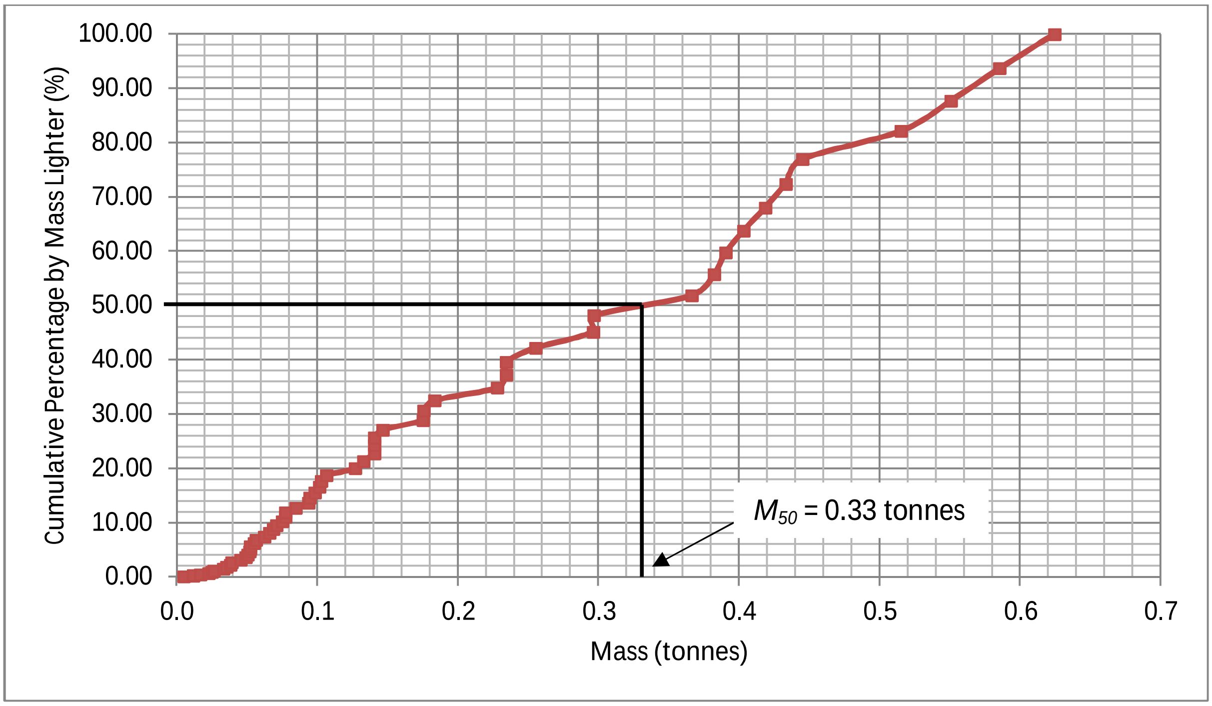


Figure 6.9: Mass distribution curve for riprap boulders at Site 7 (RR 7B).

The associated D_{n50} is determined by Equation 6.3, which is simply a rearrangement of Equation 6.2.

$$D_{n50} = \left(\frac{M_{50}}{\rho_r}\right)^{1/3} \quad \text{Equation 6.3}$$

Where D_{n50} is the median D_n value of rock grading given by 50% on the mass distribution curve. The obtained M_{50} and D_{n50} values are utilised for riprap stability equations in the following section.

Theoretical Stability of Riprap

Through continued research and experience the formulas concerning armour unit stability and performance of rubble mound structures have continued to develop and change over the past fifty years. The most commonly known is the Hudson formula (1959), which was successfully implemented by Levebvre *et al.* (1992, 1995).

$$H_s = 0.79 \left[\frac{1}{\rho_r} M_{50} \, K_D \, \Delta^3 \, \cot \alpha\right]^{1/3} \quad \text{Equation 6.4}$$

Where K_D is the stability coefficient and α is the slope angle (°) of the structure from horizontal. A K_D value of 2.2 has been applied for this study. At each site the slope of the rock revetment was measured using a clinometer. The average of three measured readings was taken. Δ is the relative buoyant density of rock and is determined by:

$$\left(\frac{\rho_r}{\rho_w}\right) - 1 \quad \text{Equation 6.5}$$

Van der Meer (1988) formulated two equations regarding the stability of rubble mound structures under the influence of either plunging or surging (non-breaking) waves. It was noted in Section 3.4.4 that Lake Pukaki is characteristic of short steep plunging waves. Thus, the Van der Meer (1988) equation associated with plunging waves has been utilised, which is shown below:

$$\frac{H_s}{\Delta \, D_{n50}} = 6.2 \, S^{0.2} P^{0.18} N^{-0.1} \xi_o{}^{-0.5} \quad \text{Equation 6.6}$$

Where S is the damage level, P is the permeability coefficient and N is the number of waves. The number of waves is difficult to estimate but Equation 6.6 is valid for N = 1000 to N = 7000. A value of N = 7000 should provide to be a conservative estimate in terms of stability (CERC, 1995). P varies from P = 0.1 for a riprap revetment underlain by an impermeable slope, to P = 0.6 for a riprap revetment with no core material. Van der Meer's (1988) theory was that a structure with higher permeability has a greater potential for dissipating wave energy and hence is more stable. The revetments at Lake Pukaki are either founded on mixed sand and gravel beach sediments or a reno mattress, so P = 0.5 has been utilised. A damage level of S = 2 is used for all the revetments. This damage level corresponds to a rock armour structure, with a slope between 1:1.5 (33.7º) and 1:3 (18.4º), which has undergone minimal damage, where only a few armour units (0 – 5%) have been displaced. The stability coefficient (K_D) used for the Hudson formula in Equation 6.4 also implies a minimal damage level with only 0 – 5% displaced rocks.

Following on from work by Vidal *et al.* (2006), Etemad-Shahidi and Bali (2011) refined Van der Meer's (1988) equation for armour unit stability by removing the N variable as it had been noted that it is hard to predict the amount of waves a structure has experienced within its operational lifetime. H_s was also replaced with H_{50}, which is the average wave height of the fifty highest waves reaching a structure in its useful life.

$$\frac{H_{50}}{\Delta\, D_{n50}} = 4.18\, P^{0.16} S^{0.18} {\xi_o}^{-0.53} \qquad \text{Equation 6.7}$$

The surf similarity parameter (ξ_o) was defined earlier in Equation 3.5. Both the Van der Meer (1988) and the Etemad-Shahidi and Bali (2011) equations were intended for armour unit stability on breakwaters. Due to breakwaters being located offshore in deeper water the breaker type defined by ξ_o is determined by the slope angle of the breakwater as opposed to using the angle of the beach profile. This approach was also employed for this study as the revetment slope was used to calculate the associated ξ_o value. When lake levels are at the upper end of the operating regime the revetments backing the shoreline are partially submerged, therefore, wave breaking is primarily influenced by the slope angle of the structure. The ξ_o values calculated here, for the purposes of riprap stability, are much greater to what was specified in Section 3.4.4 but are still considered as plunging waves.

Using the actual M_{50} and slope at a given site, the H_s and H_{50} thus calculated should theoretically represent the maximum H_s and H_{50} that a given riprap could withstand without damage; in other words, it represents a stability threshold (Levebvre *et al.* 1992). The stability threshold for the assessed rock revetments have been calculated using Equations 6.4, 6.6 and 6.7, and are summarised in Table 6.3.

The maximum wave statistics for each site are also included in Table 6.3. The strongest northerly wind event that produced winds averaging 24.11 ms^{-1} on the 24/09/10, which was previously discussed in Section 4.5, has been utilised to estimate the maximum H_s values for comparison with the Hudson and Van der Meer stability thresholds in Table 6.3. Though, it should be noted that on the 24/09/10 the lake level was situated at a low elevation of 526.91 msl so the resultant waves would have not interacted with the revetment structures but only on the lower foreshore. For this reason maximum H_s values have also been formulated from the strongest wind event during high lake levels (>532 msl), as it is at these elevations when the revetments are partially submerged. This northerly event was on the 19/01/11, where winds averaged 12.57 ms^{-1}. H_{max} values have been determined from the associated H_s values via the relationship mentioned prior in Section 3.4.1, where $H_{max} = 1.86\ H_s$. These H_{max} values are used to compare with the Etemad-Shahidi and Bali (2011) stability threshold H_{50}.

Discussion of In-situ Gradation Results

Aside from the high expected Etemad-Shahidi and Bali H_{50} values, the stability thresholds implied by the Hudson and Van der Meer formulas (H_s) show a great deal of variance. The Van der Meer H_s values are on average 28% greater than the Hudson H_s figures. The Hudson formula has been widely used because of its simplicity; however it has its limitations. The stability coefficient (K_D) value is supposed to represent numerous variables and is generally obtained through experiment (Kamali and Hashim, 2009).

Table 6.3: Rock revetment riprap gradation, stability thresholds and maximum wave experienced. The stability thresholds represent the maximum H_s and H_{50} that a given riprap could withstand without damage. H_s is significant wave height, H_{50} is the average wave height of the fifty highest waves, H_{max} is maximum wave height α is the revetment slope angle, D_{n50} is the median nominal diameter (D_n) of rock grading, M_{50} is the median mass of rock grading and ξ_o is the surf similarity parameter.

Site	**α (º)**	D_{n50} (m)	M_{50} (tonnes)	ξ_o	Hudson (H_s) (m)	Van der Meer (H_s) (m)	Max H_s (m) lake level >532 msl 19/01/11	Max H_s (m) 24/09/10	Etemad-Shahidi and Bali (H_{50}) (m)	Max H_{max} (m) lake level >532 msl 19/01/11	Max H_{max} (m) 24/09/10
1 a	19	0.59	0.53	1.93	1.38	1.76	0.45	1.05	2.82	0.84	1.95
1 b	19	0.65	0.71	1.93	1.52	1.94	0.45	1.05	3.11	0.84	1.95
2	27	0.61	0.6	2.58	1.26	1.58	0.58	1.32	2.50	1.08	2.46
4 a	22	0.56	0.45	2.26	1.24	1.55	0.68	1.54	2.47	1.26	2.86
4 b	22	0.58	0.52	2.26	1.3	1.61	0.68	1.54	2.57	1.26	2.86
7 a	22	0.65	0.71	1.97	1.43	1.92	0.87	2.05	3.08	1.62	3.81
7 b	22	0.53	0.39	1.97	1.17	1.57	0.87	2.05	2.52	1.62	3.81
8 a	21	0.75	1.1	1.89	1.68	2.26	1.09	2.35	3.63	2.03	4.37
8 b	23	0.68	0.81	2.08	1.49	1.96	1.09	2.35	3.13	2.03	4.37
9	24	0.93	2.06	2.19	1.99	2.61	1.09	2.32	4.16	2.03	4.32
11 a	30	0.58	0.52	2.93	1.15	1.41	0.83	1.67	2.22	1.54	3.11
11 b	31	0.63	0.65	3.05	1.22	1.50	0.83	1.67	2.37	1.54	3.11

The Van der Meer formula is able to account for variables such as wave height, wave spectrum shape, porosity (permeability) of the structure, damage level (loss of stability), and sea state duration. This is the most likely explanation for the noticeable difference between the two formulas as there is a heavy reliance on K_D to embody many individual parameters, whereas the Van der Meer formula is able to manage all of these separately. Another difference is that the Hudson formula takes into account the mass of each block (M_{50}) as opposed to the size (D_{n50}) used by the Van der Meer formula. The mass of each armour unit was estimated by Equation 6.2 and assumes that the blocks exhibit a cubical shape. This is not entirely correct as the riprap boulders usually have an irregular nature, being sub-rounded to sub-angular. Lefebvre *et al.* (1992, 1995) multiplied Equation 6.2 by a constant of 0.56 to account for the irregularity of armour units. This constant was not applied for this study as the characteristics of the rock armour analysed by Lefebvre *et al.* (1992, 1995) were not highlighted; this includes the rock source, shape and specific density.

Larger boulders were measured along the southern shore at Sites 8 and 9, where D_{n50} ranged from 0.68 to 0.93 m. This has resulted in both of these sites acquiring relatively large stability thresholds. The remainder of the assessed revetments around Lake Pukaki consisted of smaller sized material, with D_{n50} ranging between 0.53 to 0.65 m. By comparing the wave height values the revetments are able to resist for Sites 1 and 11 it is also apparent the affect the slope angle has on the stability threshold. Both these sites have similar D_{n50} values, though the revetment at Site 11 is steeper than the one at Site 1, leading to a lower stability threshold for Site 11.

It would have been preferable to examine the entire wave spectrum that the revetments have experienced during their operational lifetime. However, it was concluded in Chapter 3 that the use of wind station data located outside the vicinity of Lake Pukaki is unreliable for wave prediction. That is why only the wind data collected during this study period is considered suitable for wave prediction. The discussion of the maximum wave heights (H_s, H_{50}) in relation to the estimated stability thresholds is analysed in further detail in Section 6.6.

6.6 Comparison of Visual Observations and the Theoretical Stability

In the study of Lefebvre *et al.* (1995) the theoretical stability of the riprap, implied by the Hudson formula (1959), was evaluated by small scale modelling within a laboratory. The results obtained within this study were not correlated with any laboratory investigation but only what was observed in the field.

The theoretical stability determined by Equations 6.4, 6.6 and 6.7 mainly concerns the 4th rule set by the US Army Corps of Engineers, which regards the individual rock weight and density. According to the three stability thresholds, all of the revetments performed well for the 19/01/11 wind event when lake levels were high. The waves produced during this period are likely to be the largest experienced by the revetments during the timeframe of this study. The only revetment that failed to abide to the 4th rule was at Site 3. As it has been mentioned prior, the in-situ gradation of the riprap was not measured there so the theoretical stability cannot be confirmed. Some rock armour was observed to have become dislodged and scattered at Sites 4, 7 and 11, however the general integrity of the structures were not threatened.

Most of the revetments have been augmented in the last three to six years, apart from Sites 3 and 9. This is a reflection of the current performance of these structures observed in the field, with the exception of Site 9. As opposed to being backed by glacial till, like many of the other revetments, the presence of a gabion wall (behind) and reno mattress (below) at Site 9 aids in the stability of the revetment. It provides an encompassing permeable surface for the revetment, which leads to a reduction of hydrostatic pressures potentially causing uplift beneath the riprap. The rock armour at Site 9 was also the largest measured (D_{n50} = 0.93 m) across all the revetments.

The waves produced by the strong northerly wind event on the 24/09/10 did not interact with the revetments because of the low elevation of the lake level at the time. However, this event does demonstrate the maximum potential wave heights could reach during high lake levels. It seems the structures subject to the longest fetch lengths, especially along the southern shoreline, are more likely to fail during a northerly storm event of this magnitude (wind speeds of 24.11 ms^{-1}). Only the revetment at Site 1 is predicted to withstand the maximum potential waves produced by the 24/09/10 event, across all stability equations. Site 1 is exposed to the shortest fetch for northerly winds and thus experiences the smallest waves of all the sites. The revetments at Sites 2, 4 and 9 would perform satisfactorily according to the Van der Meer stability threshold but not by what is predicted by the Hudson stability threshold. The theoretical stability of Site 2 in reality would be much higher than what is predicted due to the presence of a steel rope and rail fence. This retaining feature adds further stability to the riprap slope and allows for smaller sized armour units to be positioned there.

Overall, it is difficult to determine the reliability of using Equations 6.4, 6.6 and 6.7 for assessing the stability of the rock revetments at Lake Pukaki. All of the riprap, where the in-situ gradation was measured, has performed well both theoretically and from field observations. The point of failure for each individual revetment is still unknown as there is a significant amount of variance between the three stability thresholds. The in-situ gradation was not measured for any severely damaged riprap at Lake Pukaki so the occurrence of failure could not be validated. To confirm the appropriateness of three stability equations it is recommended that the in-situ gradation of riprap be measured following a specific storm events if damage ensues. It is under these conditions when all the parameters leading to instability can be determined, including H_s and D_{n50} at failure. Ultimately, the Hudson and Van der Meer formulas are better suited for the Lake Pukaki setting as they take into account the significant wave height (H_s), which is well known and easily obtainable parameter. Considering the wave form, the Van der Meer formula provides a good representation of steep plunging waves, which are characteristic of an alpine lake. Accordingly, it is preferred over the functionality of the Hudson formula. It was initially thought that the Etemad-Shahidi and Bali equation would provide to be more useful than the Van der Meer design formula as it did not involve estimating the number of waves (N) a revetment had experienced since in operation. However, the wave height parameter H_{50} that the Etemad-Shahidi and Bali formula utilises is a difficult value to determine. All of the wind data covering the structures lifetime is required to predict H_{50} via hindcasting methods. This information is not easy to obtain especially due to the inaccuracy of off-site wind records. The reliability of using the Etemad-Shahidi and Bali formula is subjective for the Lake Pukaki setting as its associated stability thresholds are very high.

6.7 Conclusion

The effectiveness of riprap used for shore and bank protection structures at Lake Pukaki varies, with different degrees of success evident. It should be recognised that much of the work has relied on smaller than desired rock size (Opus, 2009). Under the current wave climate, during this study period (July 2010 – February 2011), the majority of the assessed revetments seem to be performing well. The revetment at Site 3 seems to be the biggest concern in terms of performance based on field observations. The poor foundation conditions of the glacial till backshore and the deterioration of the rail and rope retaining fence near the toe has lead to the instability and failure of the protection works at Site 3. Extensive erosion was also noticed along and behind the northern edge of the revetment at Site 7 and on the southern side at Site 2.

The Hudson and Van der Meer formula seem to predict respectable stability thresholds that agree with what was observed in the field. Theoretically the maximum wave height (H_s) that each individual revetment is able to withstand, based on their in-situ gradation, is not exceeded during the storm conditions (wind speeds of 12.57 ms^{-1}) produced on the 19/01/11 when lake levels were high. Few boulders were seen scattered at Site 4, 7 and 11, though the general integrity of the revetments were still intact, which was backed up by the theoretical evaluation. The movement of boulders at Sites 4 and 7 has lead to the exposure of large voids spaces within the riprap and failure according to the 6th rule.

A high lake level coinciding with storm conditions is shown to have a major influence on riprap stability. It is during this time that riprap has to withstand large, steep plunging waves, characteristic of Lake Pukaki. If the assessed northerly storm event, with wind speeds averaging 24.11 ms^{-1}, was to occur during high lake levels the revetments subject to the longest fetch lengths, especially along the southern shoreline, are more likely to fail. It is predicted that the revetment at Site 1 would perform satisfactorily but Sites 7, 8, 11 and potentially Site 4 would endure stability issues due to the resultant waves.

Chapter 7

DISCUSSION

7.1 Introduction

This chapter serves as a discussion chapter and assesses the suitability of the erosion control structures in use at Lake Pukaki. The performance of revetment structures utilising riprap was assessed in Chapter 6. The environmental factors responsible for accelerated erosion rates along the shoreline (possibly leading to structure) failure, are identified and discussed in this chapter. This involves examining the geological character of the backshore, in terms of its resistance and stability, and the wave characteristics at particular sites, in relation to their potential energy. Shoreline erosion was also produced by the interaction of shoreline protection structures and natural processes. Recommendations for achieving erosion control are outlined, which involves the option of relocating assets. A number of issues facing the managers of this lakeshore environment are also presented.

7.2 Theoretical Evaluation of Required Riprap

Theoretical Approach

The use of the Hudson and the Van der Meer stability formulas have provided to be reliable for assessing the performance of the revetments in use at Lake Pukaki. When failure was observed in the field concerning under-sized riprap boulders, it was generally backed up by the theoretical evaluation. For this reason, the theoretical evaluation has been used to determine the minimum rock size (D_{n50}) required for the revetments according to the examined wave environment at Lake Pukaki.

The maximum estimated significant wave height (H_s) within the study timeframe (late July 2010 to early February 2011), was produced by a northerly storm event on the 24/09/10, with average wind speeds of 24.11 ms^{-1}. As the study period covered the windiest months, known to be spring and summer at Lake Pukaki (Kirk, 1988; Allan, 1991), the wave heights predicted from the storm event on the 24/09/11 are potentially the largest on an annual basis. Thus, the associated H_s can be considered as the design wave height (H_d) for the theoretical evaluation, which on average, is reached or exceeded once every year.

With the possibility of the design wave height (H_d) being exceeded on more than one occasion it is important to implement an appropriate factor of safety (FoS) in the design (Burcharth and Hughes, 2003). A FoS, also known as a safety factor, is used to allow for a structure to function accordingly beyond expected loads and tolerates uncertainty in the design process. For coastal engineering:

$$\text{FoS} = \frac{\text{Material Resistance}}{\text{Design Load}}$$

Where in this case the design load is H_d and the material resistance is every other variable in the Hudson and Van der Meer equations, including the revetment slope angle (α), D_{n50}, the permeability coefficient (P) and the buoyant density (Δ).

Within the literature there is a lack of recommended FoS concerning the stability of revetments comprised of riprap material. Burcharth and Hughes (2003) suggested that a minimum FoS = 1.1 be used regarding the stability of riprap blankets used for river bank protection and a FoS = 1.5 is intended for the design of piles in a coastal environment. Compared to the riprap revetments at Lake Pukaki, riprap blankets used for river bank protection experience different hydraulic loads whereas concrete piles are comprised of different material properties. However, the FoS criteria recommended by Burcharth and Hughes (2003) are intended for structures that exhibit hydraulic loads via waves and/or currents. Therefore, the two mentioned FoS values have been implemented for the theoretical evaluation determining the minimum stone size (D_{n50}) for riprap at Lake Pukaki.

Table 7.1 presents the theoretical D_{n50} size, according to the Van der Meer formula (Equation 6.6), required for riprap at a range of sites around Lake Pukaki. It was noted in the previous chapter that the Van der Meer formula provided a better representation of lake waves as opposed the Hudson formula, as it considered the wave shape at breaking, which for Lake Pukaki is characteristic of steep plunging waves. The same variables have been applied that were used previously in assessing the stability thresholds (H_s) in Section 6.5.2, which includes P = 0.5, S = 2 and N = 7000.

Table 7.1: Theoretical evaluation, using the Van der Meer formula (Equation 6.6), of required D_{n50} (m) for riprap on revetments at several sites around Lake Pukaki. α is the revetment slope angle, $\alpha_{adjusted}$ is the adjusted revetment slope angle, FoS is the factory of safety and D_{n50} is the median nominal diameter (D_n) of rock grading.

Site	Slope (º)		Max H_s (24/09/10)	In-Situ D_{n50} (m)	Theoretical D_{n50} (m) FoS = 1.1 - 1.5	
	α	$\alpha_{adjusted}$			A	$\alpha_{adjusted}$
1	19	19	1.05	0.62	0.39 - 0.53	0.39 - 0.53
2	27	22	1.32	0.61	0.57 - 0.77	0.50 - 0.69
4	22	19	1.54	0.57	0.61 - 0.84	0.55 - 0.75
7	22	18	2.05	0.59	0.76 - 1.04	0.68 - 0.93
8	22	18	2.35	0.72	0.88 - 1.20	0.79 - 1.07
9	24	18	2.32	0.93	0.91 - 1.24	0.78 - 1.06
11	30	25	1.67	0.61	0.76 - 1.04	0.68 - 0.93

Discussion of Results

Site 1 still seems to perform well under maximum predicted conditions when a FoS of 1.1 to 1.5 is applied. On average, for design purposes utilising a FoS = 1.1 requires an increase of D_{n50} of 21% and a FoS = 1.5 involves an increase of D_{n50} of 53%. The increase of rock size (D_{n50}) required for the riprap is greater for sites situated further southwards. This is inherently due to a uniform D_{n50} size (0.57 – 0.72 m) being utilised at all sites and wave heights amplifying down the southward stretch of the lake. This, however, excludes the riprap for the revetment at Site 9, which is very large (D_{n50} = 0.93 m) in comparison to the other revetments.

To assess the effect the slope angle (α) has on the theoretical stability of the revetments, adjusted slope angles ($\alpha_{adjusted}$) were included in Table 7.1. A revetment should have a gentle a slope as possible, though it is limited by the available space along the beach front and it is preferred to be located as landward as possible to minimise interference with the nearshore process regime (Reeve *et al.* 2004).

Depending on site conditions, such as the beach profile and the dominant wave approach angle, the revetments α were adjusted accordingly. Site 4's riprap angle was adjusted to the same angle as Site 1, which due to its theoretical performance remains unchanged. All the other revetments were subject to a decrease in α of ~20%. The lowering of the structures α has lead to an 11% reduction in D_{n50} required for design. Variance of the slope angle has a bigger influence on the rock size (D_{n50}) further south, as the effective fetch increases. It is interesting to note that the recommended D_{n50} is the same for Sites 7 and 11. This further highlights the effect H_s and α have on the stability of a revetment structure.

Reducing the slope angle of a revetment not only requires smaller rock sizes but also causes waves to break further offshore, as breaker height is a function of water depth. Due to the steep nearshore profile of Lake Pukaki, wave breaking is controlled by the slope angle of the protection structure.

The range of lake levels is a significant factor in regards to the slope angle. Due to the limited amount of area behind the current structures, to effectively decrease the slope angle the structures must extend down slope. This leads to problems during low lake levels as large waves are able to undermine the toe of the revetment structure. Lowering the slope angle of the revetment at Site 9 is also concern as it likely will extend beyond the reno mattress it is founded on. The presence of a reno mattress aids in the protection of potential scour beneath the toe of the revetment at Site 9. The toe of the revement will thus be more susceptible to under mining if it is extended beyond the reno mattress. This may require the reno mattress to be extended or the foundations dug in beach slope.

This section has highlighted the importance of applying a Factor of Safety (FoS) within the design of shore protection structures, in particular for revetments utilising a riprap armour layer. It is preferred that a FoS =1.5 is used in the design over a FoS = 1.1 as it allows for a larger margin of variability to exist within the wind and wave climate. It is assumed that increasing the rock armour size, especially for sites exposed to the longest fetch, and lowering the slope angle of the revetments has a major influence on the structural integrity of the riprap. This option is discussed in more detail in Section 7.4 concerning future management of shoreline erosion at Lake Pukaki.

It should be noted that the Hudson and Van der Meer stability formulas only take into account one type of failure. This type of failure concerns Rule #4 according to CERC (1984b), and is a matter of an individual armour units weight and its placement. There are other factors that lead to the failure of shore protection structures, aside from the hydraulic stability of individual boulders. These are further discussed in the following section.

7.3 Environmental Factors Leading to Erosion and Protection Failure

It was concluded in Chapter 6, that the majority of revetments at Lake Pukaki are performing adequately under the examined wave environment, with the exception of Site 3. However, particular sites did show signs of failure, concerning one or more of the 'six rules' used by CERC (1986b). The potential energy available to cause erosion and instability of protection works needs to be understood. The environmental factors inducing shore protection failure and accelerated rates of erosion along the shoreline are outlined below. The contrary effects of protection structures on erosion are also discussed.

7.3.1 Lake Level Influence

It has been identified by this study and many previous lacustrine studies (for example Davidson-Arnott and Keizer, 1982; Lorang *et al.* 1983; Lawrence, 1995; Kirk *et al.* 2000; Pierce, 2004) that the lake level elevation is a controlling factor for erosion and shore protection performance. Erosion control structures backing the shoreline are only influenced by hydraulic loads during high lake levels. It was shown during a 1995/96 flood event at Lake Hawea that prolonged periods water levels exceeding the elevations of revetments contributed to their failure (Kirk *et al.* 2000).

The short-period high-magnitude storm events, eventuating from a strong north/northwest wind flow, that coincide with high lake levels tend to cause the most significant erosion along the shoreline at Lake Pukaki (Kirk, 1988; Allan, 1991), potentially leading to structure failure. It has been documented by Opus (2009) that maintenance of shore protection structures is usually carried out following high energy storm events.

Chapter 5 examined the frequency and occurrence of high magnitude, short term storm events that combined with high lake levels. The wave power (*P*) estimates demonstrate that there is a high percentage of storm activity during December 2010 and February 2011. The higher magnitude events predominantly occurred during December; however lake levels during this time were still below the maximum operating range. A storm event, exceeding 500 $Nms^{-1}m^{-1}$ of wave crest, which endured for five days (30/12/10-3/01/11), was expected to have a significant impact on the backshore as it was when lake levels were at their highest elevation of 532.38 to 532.87 msl. Maximum wave runup heights ($R_{2\%}$) were also estimated to exceed the base of the cliff at Site 8 a total of seven days during January 2011. Pierce (2004) expressed that high amplitude waves at low water levels are of much less significance than somewhat smaller waves at high water levels as the full force of the waves affects the upper beach. Therefore, it is during January 2011 that a high amount of erosion is expected to occur within the backshore region, not only at Site 8 but along the entire stretch of the Lake Pukaki shoreline. In contrast to the southern shoreline, the beaches on the eastern and western shores are narrower and steeper, which leads to an increase in wave runup elevations. At Site 4 over the course of January 2011 waves reached above the base of the cliff for 16 days (52% of the measurement period), which is more than twice as much than was predicted at Site 8. Due to the exposure to northerly and southerly winds, sites on the eastern and western shores endure a high frequency of waves acting on the cliffs backing the beaches.

Overtopping of structures is more pronounced during high lake levels. The only revetments at Lake Pukaki that were identified as at risk of overtopping were Sites 2 and 7. Even if it is at risk, the height of the revetment at Site 2 cannot be easily raised as it is already situated at road level. There is potential for water spray to reach the road but according to the 5th design rule this is acceptable. The top of the revetment at Site 7 is situated only ~1 m above a high water level of 532.4 msl. According to the predicted maximum H_s in Table 7.1 it is likely to be overtopped. This assumption is backed up by what was observed in the field where erosion was evident behind the revetment and has resulted in failure in accordance to the 5th rule. In terms of protecting assets behind the backshore, Site 2 is still the biggest concern as it is adjacent to the road, whereas the revetment at Site 7 is ~10 m from the roads edge (Braemar Road).

7.3.2 Steep Nearshore Profile

Lake Pukaki is characteristic of steep plunging waves, which are common on South Island alpine lakes (Allan, 1998; Dawe, 2006) and generally erosive in nature. The steep nearshore profile determines the wave form but also allows waves to break in close proximity to the shoreline. The resultant swash zone is narrow and very close to the shoreline. Strong longshore currents (0.151 – 0.197 m^3 h^{-1}) are typical of the eastern and western shores, forming within the swash zone as waves approach at oblique angles to the beach orientation. Single (2006) believes that the lowering of the nearshore shelf has occurred due to strong longshore currents on the eastern shore. For the Great Lakes of Northern America, in areas where longshore currents remove coarse sediments, the vertical lowering of the profile is the primary control for bluff recession (Davidson-Arnott, 1986). Longshore currents have the ability to form beach steps (Figure 2.8) and remove beach sediments immediately in front of protection structures. The continuing erosion of the nearshore profile exposes shore protection structures to higher wave energy and potential undermining of the foundations. According to the Opus shoreline investigations (2009), strong longshore currents at Site 12 have created several problems for the current erosion control structures in place. A combination of high turbulent flows directed alongshore and entrained coarse grained sediment has lead a high amount of repair work being conducted for the wire mesh used for the reno mattress, which extends beyond the backshore and covers the beach slope.

7.3.3 Glacial Till Backshore

A high proportion of the Lake Pukaki shoreline is surrounded by steep over-consolidated bluffs. In fact, all of the study sites (shown in Figure 2.9) utilised for this research, apart from Sites 2 and 7 are backed by steep cliffs. Tekapo Till is the most widely distributed deposit about the shores of Lake Pukaki. It comprises of a high percentage of silt, is easily eroded and has poor resistance to wave action. The suggested mechanism for erosion in till is abrasion by surficial sediments and hydraulic pressure due to wave action, which is aided by softening of the till, likely due to cyclic fatigue failure (Davidson-Arnott, 1986).

The unstable glacial till backshore at Site 3 was the primary factor causing failure of the revetment. Settlement of the underlying material at Site 3 has lead to slumping and the formation of several scarps within ~1 m of the roads edge. The instability of the foundation material is the main reason why the in-situ gradation was not carried out at this.

According to Opus (2000) the material is susceptible to rapid weathering when saturated. The impact of raising the groundwater level brought on by high lake levels can cause the lower section of the bluffs to become fully saturated, which can ultimately lead to instability. Precipitation also had an effect on cliff weathering and erosion through saturation and slumping. Erosion of the cliffs along the western shore has been caused by a combination of wave action at the cliff base and rainfall causing instability of the cliff face (Single, 2006). Groundwater levels and precipitation were not measured for this study, in terms of assessing the stability of the glacial till cliffs, though it does present an area for further research.

7.3.4 Adverse Effects of Shoreline Structures

Kirk *et al.* (1996) stated that revetments and other hard shore protection structures have the contrary ability to refer erosion to nearby areas. It is often the interaction of nearshore processes with poorly designed structures that leads to enhanced erosion. The effect of shore protection structures is often felt on adjacent shores as well as on those that the structure is designed to protect (Hansom and McGlashan, 2000). It is important that structures implemented for the purposes of decreasing the rate of erosion incorporate the ‘six rules’ of design outlined by CERC (1986b). Failing to abide to the ‘six rules’ can cause a structure to function in opposition to what it was intended.

A series of discontinuous revetment sections can lead to localised intensive scouring of the exposed sections of backshore between them (Kirk *et al.* 1996). This is particularly significant at Site 4 opposite Tasman Downs homestead, shown in Figure 7.1, where four individual revetment sections were positioned along the backshore. The hard rock structures on either side concentrate wave energy onto the weaker glacial till, resulting in a zone of enhanced scour causing cusps to develop in the cliff face. The general consensus to mitigate this is extending the revetment to fill in the exposed sections. However, this only shifts the erosion problem further down the shoreline.

Figure 7.1: Scour and pockets of erosion evident along an exposed backshore section between two revetment structures at Site 4, looking east. Scattered riprap is more pronounced for the revetment to the left of the picture, which also consists of irregular shaped concrete blocks.

The formation of a gap behind the northern edge of the revetment was observed at Site 7, as shown in Figure 6.5. This type of failure concerns the 2nd rule according to CERC (1984b). This has developed due to outflanking and has lead to the structure becoming disconnected with the backshore. The gap between the structure and backshore will continue to grow as accelerated rates of erosion is concentrated in this region. Outflanking was also observed at the southern end of the Site 2 revetment. The cause of erosion here is likely due to waves resulting from southerly winds. Scour was also evident on the western edge of the revetment at Site 8, shown in Figure 6.4.

The amount of erosion that was previously present along the cliff side before the revetment was constructed is not known. Therefore, the actual effects imposed by the structure on the adjacent backshore cannot be quantified.

Aside from accelerating erosion rates between the backshore and the structure, some erosion control structures unintentionally restrict littoral drift currents and starve down fetch beaches of sediment. This is with the exception of groynes, which is the main concept of its design. Beach sections adjacent to the rock revetments did not show signs of deprivation in terms of sediment. The interference of longshore currents caused by the revetments must be very minimal and is understandable as most are constructed at steep angles (19-31°).

7.4 Recommendations for Erosion Control and Future Management

For shoreline erosion management to be successful at Lake Pukaki, a number of fundamental issues need to be addressed. A firm understanding of nearshore processes is necessary for the designing and planning of shoreline erosion control techniques. Knowledge of other environmental factors including lake level seasonality, beach sediment behaviour, groundwater conditions and backshore material properties are required. All the available options for shoreline protection for a specific site need to be assessed in terms of their related costs and their effectiveness. Whichever lakeshore protection strategy is adopted the design and construction must be comprehensive. For hard engineered structures the design parameters need to incorporate the 'six rules' used by CERC (1986b).

Shore inspections and surveying should continue on an annual basis or following specific storms, as done so in the past. Continued monitoring evaluates the shoreline response to the nearshore process environment at Lake Pukaki and the analysis surrounding this can assist in refining the design and implementation for current and future shoreline erosion control strategies.

Meridian Energy should be familiar with the increased risk to the road network associated with maintaining the lake at a high level for considerable periods. Understanding that Lake Pukaki is a major contributor for the Upper Waitaki hydro-electric power scheme, and that the maximised volume of water contained within the reservoir leads to better economic stability. It should be expected that over time the demand for energy and power will increase and inherently so too will the lake level. The option of altering the regulated lake level fluctuation by changing the timing and the rate of lake level drawdown, resulting in a lower occurrence of extreme conditions is not foreseeable. For this reason, it seems there are two possible strategies available in order to continue to protect the road network flanking the Lake Pukaki shoreline. The first option is to persist in maintaining the erosion control structures that have been in operation since 1987 and to construct new structures where needed. The second option is to relocate the road further back from the shoreline. These two options are discussed in greater detail below.

7.4.1 Construct and Maintain Current Erosion Control Structures

The current rock revetments in use at Lake Pukaki have been assessed to be performing well under the current wave environment. They are considered as low cost solutions for combating the extensive shoreline erosion. Regular maintenance of these structures is included as part of their design with most of them having received repair in the past three to six years. Sourcing local materials for the riprap is a cost-effective option, reducing transport costs and long-term availability of rock is essential for maintenance. However, it has been acknowledged (Opus, 2009) that most of these structures have required considerable repair following storm events. Failure to abide to the 'six rules' has to lead to these structures having adverse effects on the nearshore processes leading to accelerated rates of erosion in the vicinity of the structures.

Lakeshore managers need to be proactive as opposed to reactive in terms of shoreline mitigation. This involves designing erosion control structures such as revetments or groynes that are able to withstand the hydraulic forces presented by the wave environment. The theoretical evaluation of the required D_{n50} for riprap was carried out in Section 7.1. Employing a FoS = 1.5 in the design involves an increase of D_{n50} of ~53% and with a reduced slope angle an increase of D_{n50} of ~42%. Reducing the slope angle of the revetments involves increasing the length it extends from the backshore. This seems a more preferred option for sites situated along the southern shoreline as opposed to the eastern and western shores. The lower slope angle will cause waves to break further offshore, requires smaller rock sizes and reduces the amount of wave runup occurring on the structures. Implementing gentler revetments on the eastern and western shores may inhibit strong longshore transport evident along these shores and starve down-fetch locations of sediment. It should also be noted that if any rock armouring such as riprap is placed on glacial till it is recommended that a gravel filter layer and a geotextile fabric is incorporated in the design to prevent the impact of wave energy on the till and to reduce the loss of fine material.

A similar approach to what Kirk *et al*. (2000) proposed for the southern shoreline of Lake Hawea could be employed at Lake Pukaki. This involves building a wider beach in critical areas by beach nourishment and constructing groynes to restrict the movement of the nourished gravel. This is a common strategy as these two techniques work well in unison. The southern shoreline of Lake Hawea is similar to the southern shoreline at Lake Pukaki, as they are both subject to large wind waves from strong northerly winds and experience longshore sediment transport predominantly westwards. The construction of groynes along the southern shoreline at Sites 7 and 8 have been noted by Opus (2009) to have performed well at trapping sediment due to longshore currents moving westwards. A large amount of sediment has built up on the eastern sides of the groynes. The construction of them however is subjective as the rocks comprising them have continually dispersed over time.

Though, they have achieved what they were originally intended, as according to Opus (2009), these groyne structures were initially set up as trials. Riprap was dumped and not carefully placed during construction to keep costs low. Properly built groynes should ultimately perform better compared to the predecessors. If beach nourishment were used, sediment of similar size and shape compared to the existing beach sediments would be needed.

7.4.2 Relocating Assets

The economic cost likely to be spent mitigating the shoreline at Lake Pukaki in the long term, (i.e. over the next 10 years) needs to be quantified. The option of relocation of infrastructure away from the shoreline therefore can be examined. It is generally an expensive method but may benefit in the long term. This option puts more of a price on the lakeshore itself as opposed to what is situated behind the backshore. It also preserves the recreational amenity of the beach front. The demand for power and energy is only going to increase over time, and inherently so will the need to consider options for management of the lake level.

Relocation could be a viable for the eastern and southern shore. Gently sloping farmland is locating behind SH 8 (southern shore) and beyond Hayman and Braemar Rd (eastern shore). It is however difficult to implement this strategy at several other locations around Lake Pukaki as the steep mountainous terrain limits available work space. This is the case for SH80, which is confined by the Ben Ohau range bordering the western side of the lake. This is a common problem for alpine lakes in the South Island as many of the State Highways are contained within a narrow stretch between the lakeshore and a steep mountain range.

If the option of relocating the road further back was taken the time required for the shoreline to reach an equilibrium would be unknown. It is difficult for the Lake Pukaki shoreline to reach an equilibrium profile, at any site due to a continually fluctuating water level.

Chapter 8

CONCLUSION

The main objective of this research was to assess the suitability of the erosion control structures in place at Lake Pukaki; an alpine lake in the central South Island of New Zealand with no immediate habitation around its shores, and which is subject to hydroelectric-operating range changes in water level. This thesis has followed a versatile method, involving the examination of the nearshore process environment at Lake Pukaki and its influence on the current performance of the shoreline protection structures.

The following section summarises the major findings of this research according to the initial detailed aims set out in Chapter 1.

8.1 Thesis Aims Revisited

1. *To investigate the wind system around Lake Pukaki as wind-driven wave action is the most significant natural erosive force acting on the shoreline.*

Two fully automated weather stations were installed on the eastern and southern shore to investigate the wind system at Lake Pukaki. The use of wind records in close proximity to the lakeshore that are representative of the Lake Pukaki wind-wave regime, are highly regarded for use in wave hindcasting models.

Analysis of the wind regime on Lake Pukaki has identified two principle wind directions for wave generation. In an alpine environment, such as Lake Pukaki, the winds at any particular site are highly localised and the surrounding topography exerts a major control. The influence of topography channels airflow down the long north-south axis of the lake. Waves that cause the most significant erosion along the shoreline are associated with winds primarily from the northwest/north/northeast sector originating from the Tasman Valley.

Strong gusty winds (>5.7 ms^{-1}) were restricted to the northerly direction. Winds from the south/southwest are classed as of secondary importance in terms of wave development. The variation in observed wind speeds across the long axis of the lake is due to the occurrence of north/north-westerly foehn winds oscillating in the Pukaki Valley. This explains why stronger northerly winds may be restricted to the northern half of the lake.

The utilisation of wind data, dating back to early 2003, was to investigate the possibility of a storm event coinciding with a high lake level. Three CliFlo stations in the near vicinity of Lake Pukaki were used for this analysis. It was shown that the resultant wind patterns for the CliFlo stations were highly influenced by nearby topography and were not representative of the Lake Pukaki wind regime. Elevation adjustments were also carried on these stations for possible wave hindcasting execution, though they were later proven unreliable.

The east weather station data has been implemented for the majority of wave hindcasting, simply because no adjustments were necessary, apart from the use of a stability correction factor (R_T).

2. *To measure wave characteristics using accurate wave recording devices, during high-magnitude storm events.*

The use of the RBR XR-620 pressure sensor in this study marks the first occasion wave statistics have been measured via instrumentation at Lake Pukaki. With the ability of sampling at 6 Hz, the XR-620 was sufficient at capturing the wave form.

The mean significant wave height (H_s) identified for this study was 0.53 m, while the maximum recorded wave height was 1.84 m, which was produced by winds averaging 13.06 ms^{-1}. When compared with studies conducted by Pickrill (1976), Allan (1998) and Dawe (2006) on other South Island alpine lakes, Lake Pukaki's wave spectrum is capable of achieving much greater wave heights and large wave periods (1.8 – 3.8 s). The associated wave lengths (L_o) are also very large in comparison, averaging 13 m, which have developed primarily due to a longer effective fetch length at Lake Pukaki. The values obtained for the spectral bandwidth parameter (ε) suggest that Lake Pukaki is associated with swell conditions, which is the opposite of what was initially expected and observed in the field. This is potentially a shortcoming with using a pressure sensor, such as the XR-620, for the recording of high frequency waves in a restricted fetch environment.

Lake Pukaki is characteristic of steep plunging waves as with many other alpine lakes. Being highly erosional in nature, these waves are able to break very close to the shore owing to the steep gradient of the foreshore. Alpine lake waves exhibit more of a vertical energy component, as opposed to open-coast waves, as wave energy is concentrated in a narrow zone just in front of the trough as the wave breaks.

3. *To correlate measured wave statistics against modelled wave statistics to ascertain the accuracy of estimating wave characteristics along the shoreline.*

Wave characteristics could not be directly measured along the entire shoreline of Lake Pukaki, but could be modelled via wave hindcasting. The LAKEWAVE model was correlated with wave statistics measured by the XR-620 to determine the accuracy of estimating wave statistics about Lake Pukaki. The LAKEWAVE model was found to over-predict wave height and wave period at Lake Pukaki. A similar pattern was discovered by Allan (1998) when he correlated measured waves at Lake Dunstan with predicted waves determined by the NARFET model, which LAKEWAVE is based upon.

LAKEWAVE estimated FAS to occur much sooner than what was measured in the field. The time required for FAS to develop on Lake Pukaki is on average ~27% longer than what is implied by the LAKEWAVE model. Wave development within the LAKEWAVE model ceases when these conditions prevail. This phenomenon was justified on the 15/01/10 at Site 8 and on the 6/2/11 at Site 5. The measured wave heights continue to increase whereas the modelled values stay constant.

The maximum H_s and T values estimated by LAKEWAVE have been found to compare well with measured wave statistics, although they do not always occur at the same time. This finding reflects the reliability of LAKEWAVE being able to estimate maximum wave parameters over the course of a specific period or event. Predicted daily maximum wave statistics were examined in Chapter 5 regarding extreme conditions and were implemented in Chapter 6 for assessing the performance of the revetments. Most design equations that take into consideration wave parameters such as H_s or T require the maximum associated values to model the worst-case scenario.

LAKEWAVE is unable to take into account variable and gusty winds that are accelerated down valleys as it requires a constant wind blowing across the entire fetch. This is the likely reason why the model predicts FAS to occur sooner than what was measured in the field. There is a heavy reliance on the wind data supplied by the east station and that it is supposed to represent the entire wind field for Lake Pukaki. It is evident that the source of wind data is essential in terms of accurate wave prediction.

4. *To examine the frequency and occurrence of high magnitude storm events coinciding with high lake levels.*

The LAKEWAVE model has been used to describe the wave environment about the Lake Pukaki shoreline in terms of its maximum energy potential. The lake level elevation has been identified as a controlling factor for the level at which wave action is concentrated on the beach profile. It is storm conditions coincident with high lake levels that can lead to significant erosion along the backshore. Extensive erosion at the base of cliffs can result in eventual slope instability and failure.

There is a high incidence of storm activity during the summer months, average daily wave power estimates were calculated during December 2010 and January 2011 for Site 8, which is exposed to the longest fetch. In total twenty days surpassed 500 $Nms^{-1}m^{-1}$ of wave crest and ranged from 546 to 2727 $Nms^{-1}m^{-1}$ of wave crest. December 2010 was shown to have a higher occurrence of storm events than January 2011. However, it is when lake levels are at their maximum elevation that wave energy is concentrated at the upper limits of the beach profile. For this reason more extensive erosion along the backshore has likely to have occurred during January 2011.

At Site 4 wave runup intercepted the base of the cliff for 16 days during January 2011, whereas runup heights at Site 8 exceeded the base of the cliff a total of 7 days. During high lake levels, the width of the foreshore is shortened and becomes very narrow, especially on steeper beaches along the eastern and western shoreline. This has lead to an increase of runup elevation for the beaches located on the eastern and western margins of Lake Pukaki.

Longshore transport rates were estimated using the LEXSED equation, formulated by Dawe (2006), for a range of sites at Lake Pukaki during January 2011. Predicted Q_1 rates were on average higher at Lake Pukaki than what was measured at Lake Coleridge by Dawe (2006). The eastern and western shorelines are subject to a larger rate of sediment transport (0.088 - 0.197 $m^3 h^{-1}$) under longshore currents compared to the southern shoreline (0.066 – 0.083 $m^3 h^{-1}$).

5. *To assess the current shoreline protection structures used to combat the ongoing erosion along the shoreline of Lake Pukaki.*

Rock revetments are the most common erosion control structure at Lake Pukaki. A total of eight rock revetments (Sites 1, 2, 3, 4, 7, 8, 9 and 11) located at different sites about Lake Pukaki, were assessed in terms of their performance. They were evaluated based on upon visual observations and their theoretical stability. Other nearby shore protection structures that assisted in the stability of the revetments at these identified sites was also discussed.
Under the current wave climate, experienced during this study period (July 2010 – February 2011), the majority of the assessed rock revetments seem to be performing well. The revetment at Site 3 seems to be the biggest concern in terms of performance based on field observations. The poor foundation conditions of the glacial till backshore and the deterioration of the rail and rope retaining fence near the toe has lead to the instability and failure of the protection works at Site 3. Failures according to the 'six rules' set by CERC (1986b) were observed at several sites. Extensive erosion was noticed along and behind the northern edge of the revetment at Site 7 and on the southern side at Site 2. Large void spaces were exposed within the revetments at Sites 4 and 7 due to the displacement and scattering of smaller sized boulders.

Theoretically the maximum wave height (H_s) that each individual revetment is able to withstand, based on their in-situ gradation, is not exceeded during the storm conditions (wind speeds of 12.57 ms^{-1}) produced on the 19/01/11 when lake levels were high. The Hudson and Van der Meer formula seemed to predict respectable stability thresholds that agree with what was observed in the field. The Hudson and Van der Meer formulas are better suited for the Lake Pukaki setting as they take into account the significant wave height (H_s), which is well known and easily obtainable parameter. Considering the wave form, the Van der Meer formula provides a good representation of steep plunging waves, which are characteristic of an alpine lake. Accordingly, it is preferred over the functionality of the Hudson formula. The reliability of using the Etemad-Shahidi and Bali formula is subjective for the Lake Pukaki setting as its associated stability thresholds are very high. It was also difficult to obtain the H_{50} parameter from hind casted values.

A high lake level coinciding with storm conditions is shown to have a major influence on riprap stability. It is during this time that riprap has to withstand large, steep plunging waves, characteristic of Lake Pukaki. The theoretical evaluation of the required riprap size (D_{n50}) for the revetments was assessed in accordance to the maximum predicted wave heights estimated during the study period. Lake levels were low during this event on the 24/09/10, but it is supposed to model the maximum wave heights that are likely to be reached if they coincided with a high lake level. Thus, predicting the worst-case scenario for potential failure. Employing a FoS = 1.5 in the design requires an increase of D_{n50} of ~53%. This, however, excludes Site 1 as the riprap in use there is already an adequate size in relation to its effective fetch. The lowering of the slope angle of the revetments further reduced the required D_{n50} by ~11%.

6. *To identify the controlling factors in determining the success or failure of different types of shore protection around the lake.*

A number of environmental factors have been identified to have an influence on the performance of erosion control structures around Lake Pukaki. The lake level elevation is the primary factor for erosion and shore protection performance. Erosion control structures backing the shoreline are only influenced by hydraulic loads during high lake levels. The short-period high-magnitude storm events, eventuating from a strong north/northwest wind flow, that coincide with high lake levels tend to cause the most significant erosion along the shoreline at Lake Pukaki. Overtopping of structures is also more pronounced during high lake levels.

The steep nearshore profile of Lake Pukaki allows for steep plunging waves to break in close proximity to the shoreline. The resultant swash zone is narrow and very close to the shoreline. Longshore currents developing within the swash zone have the ability to form beach steps and remove beach sediments immediately in front of protection structures. The continuing erosion of the nearshore profile exposes shore protection structures to higher wave energy and potential undermining of the foundations.

Tekapo Till is the most widely distributed deposit about the shores of Lake Pukaki. It comprises of a high percentage of silt, is easily eroded and has poor resistance to wave action. The unstable glacial till backshore at Site 3 was the primary factor causing failure of the revetment. Settlement of the underlying material at Site 3 has lead to slumping and the formation of several scarps within ~1 m of the roads edge.

The groundwater level and precipitation were also noted to have an influence on cliff stability. The impact of raising the groundwater level brought on by high lake levels can cause the lower section of the bluffs to become fully saturated, which can ultimately lead to instability. Precipitation also had an effect on cliff weathering and erosion through saturation and slumping. Erosion of the cliffs along the western shore has been caused by a combination of wave action at the cliff base and rainfall causing instability of the cliff face.

Aside from environmental factors, the shoreline protection structures were also identified to have an influence on shoreline erosion. Revetments and other hard shore protection structures can refer erosion to nearby areas. It is often the interaction of nearshore processes with poorly designed structures, not abiding to the six rules' that leads to enhanced erosion. The formation of a gap behind or an exposed section between revetments leads to localised sections of enhanced scour where cusps and pockets of erosion form.

8.2 Recommendations for Future Research

This study has investigated the nearshore process regime in order to determine the appropriateness and performance of shoreline protection structures in use at Lake Pukaki. Due to the lack of research regarding nearshore processes and lakeshore management on alpine lakes this thesis has contributed to a better understanding towards deep water wave characteristics and the environmental factors leading to accelerated rates of shoreline erosion and potential shoreline protection failure. There are however a great deal of potential research opportunities in these areas and suggestions for future research are outlined below.

The examination of the oscillatory wind flow occurring along the Pukaki Valley presents an area for further research. Allan (1991) believed that the variation in observed wind speeds across the long axis of the lake was due to the occurrence of north/north-westerly foehn winds oscillating in the Pukaki Valley. This may include a more intensive study focusing on the complex wind system at Lake Pukaki by recording the variance in wind speeds at several sites situated along the long axis of the lake.

Findings regarding the spectral bandwidth (ε) have identified a flaw with using a pressure sensor for wave measurement in a lacustrine environment as they are unable to detect the higher frequency wave crests. It is recommended that the use of pressure sensors in restricted fetch environments for wave recording purposes be correlated with high sampling wave gauges. This will help determine the effectiveness and accuracy of using pressure sensors for wave measurement.

Longshore currents are proven to have a significant influence on sediment transport and erosion for narrow and deep alpine lakes. To validate the reliability of using the LEXSED formula, developed by Dawe (2006), it is necessary to acquire respective k values that are representative of the site conditions of Lake Pukaki. The k value is a coefficient. The installation of an instrument that records the direction of wave propagation would prove more accurate than the wave breaker approach angle (α_b) determined by wave hindcasting models.

In conjunction with other environmental parameters, groundwater level and precipitation were shown to be factors in determining rates of bluff recession. Much work has been concerned with this area regarding research carried out on the Great Lakes in Northern America. However, little has been done in terms of the stability of glacial till backing the shoreline for the majority of the glacial lakes in the South Island of New Zealand.

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APPENDIX 1
MATLAB (.DAT file) example

RBR XR-620 6.30 012882 (Windows: 6.08 - Minimum required: 6.05)
Host time 10/08/12 21:20:35
Logger time 10/08/12 21:20:41
Logging start 10/08/12 13:20:00
Logging end 10/08/12 15:19:59
Sample period 6Hz profiling
Number of channels = 3, number of samples = 86374, mode: Logging Complete
A03%9.4f
Calibration 1: 0.051212414000000
124.953771399999990
0.000000000000000
0.000000000000000 mS/cm
Calibration 2: 0.003370087927994
-0.000255768654588
0.000002360062264
-0.000000069320380 Degrees_C
Calibration 3: 0.132126443354000
162.714619328645990
10.429253003560000
4.190865411889000 deciBars
Correction to conductivity: 1.3500000e-004 1.0000000e+000 1.5000000e+001 0.0000000e+000
Atmospheric pressure 9.418968276 dBar, water density 0.999888, (Simplified calculation)
Latitude 0 degrees 0.000 minutes
Memory type: 6 AT45DB642D_LP
Wave burst sample rate: 6Hz
Wave burst sampling period: 01:59:59
Wave burst sample length: 7200
Logger height: 0.30
Averaging: NONE
Logger start: Start of logging
Timestamp 2010/08/12 13:00:00 at sample 1 of type: TIME STAMP
Timestamp 2010/08/12 13:00:01 at sample 3 of type: TIME STAMP

	Cond	Temp	Pres	Depth
13:20:00	0.038	7.271	10.183	0.779
13:20:00	0.032	7.271	10.182	0.778
13:20:00	0.036	7.271	10.182	0.779
13:20:00	0.037	7.271	10.182	0.779
13:20:00	0.031	7.272	10.183	0.779
13:20:00	0.036	7.272	10.183	0.779
13:20:01	0.033	7.272	10.184	0.780
13:20:01	0.037	7.272	10.184	0.780
13:20:01	0.033	7.272	10.183	0.779
13:20:01	0.033	7.272	10.182	0.778
13:20:01	0.036	7.272	10.183	0.778
13:20:01	0.0301	7.2713	10.182	0.778
13:20:02	0.0319	7.2711	10.181	0.777

APPENDIX 2
RBR XR-620 Summary Wave Statistics

All symbols defined on page xv and xvi. Average beach slope angle (β) is also included.

Location	Date	Time	H_s (m)	H_{10} (m)	H_{max} (m)	H_{rms} (m)	T_s (s)	T_z (s)	T_c (s)	$\boldsymbol{\varepsilon}$	L_o (m)	H/L	ξ_o
Site 7	12/08/10	14:20	0.09	0.12	0.25	0.06	2.47	2.41	1.38	0.74	9.05	0.010	-
		14:40	0.21	0.26	0.34	0.15	1.90	1.84	1.70	0.08	5.26	0.039	0.76
β= 8.56°		15:00	0.42	0.55	0.76	0.29	3.03	2.44	1.97	0.24	9.28	0.045	0.71
		15:40	0.73	0.91	1.20	0.52	3.82	3.15	2.03	0.56	15.54	0.047	0.69
		16:00	0.65	0.82	1.01	0.47	3.83	3.19	2.09	0.52	15.89	0.041	0.74
		16:20	0.75	0.95	1.29	0.54	4.10	3.34	2.08	0.61	17.43	0.043	0.73
		16:40	0.81	1.05	1.51	0.58	4.51	3.67	2.16	0.70	21.01	0.038	0.77
Site 8	18/12/10	13:00	0.09	0.13	0.22	0.06	1.57	1.95	1.43	0.36	5.94	0.016	-
		13:20	0.14	0.18	0.27	0.10	1.67	1.64	1.60	0.03	4.22	0.033	0.62
β = 6.46°		13:40	0.19	0.24	0.44	0.13	1.82	1.76	1.70	0.04	4.85	0.038	0.58
		14:00	0.29	0.37	0.48	0.20	2.18	2.05	1.93	0.06	6.56	0.044	0.54
		14:20	0.34	0.41	0.51	0.24	2.50	2.29	2.06	0.11	8.21	0.042	0.56
		14:40	0.37	0.21	0.52	0.26	2.66	2.38	2.09	0.14	8.87	0.041	0.56
		15:00	0.41	0.50	0.62	0.30	2.82	2.56	2.14	0.19	10.21	0.040	0.57
		15:20	0.44	0.53	0.67	0.32	2.93	2.63	2.09	0.26	10.81	0.041	0.56
		15:40	0.39	0.48	0.65	0.28	2.80	2.52	2.04	0.23	9.90	0.039	0.57
		16:00	0.36	0.45	0.65	0.26	2.90	2.58	2.07	0.24	10.37	0.035	0.61
		16:20	0.32	0.41	0.55	0.23	2.87	2.55	2.09	0.22	10.17	0.032	0.63
		16:40	0.29	0.35	0.47	0.21	2.75	2.42	2.02	0.20	9.17	0.032	0.64
		17:00	0.24	0.30	0.39	0.17	2.65	2.37	2.03	0.17	8.81	0.028	0.68
Site 8	15/01/11	12:20	0.34	0.46	0.57	0.24	3.61	3.13	2.75	0.14	15.32	0.022	0.76
		12:40	0.56	0.69	0.91	0.40	3.84	3.34	2.52	0.33	17.43	0.032	0.63
β = 6.46°		13:00	0.70	0.90	1.32	0.50	3.75	3.25	2.37	0.37	16.50	0.042	0.55
		13:20	0.78	1.00	1.31	0.55	3.90	3.26	2.26	0.44	16.57	0.047	0.52
		13:40	0.79	0.98	1.36	0.56	3.69	3.18	2.23	0.43	15.79	0.050	0.51
		14:00	0.70	0.86	1.09	0.50	3.72	3.17	2.21	0.43	15.73	0.044	0.54
		14:20	0.77	0.95	1.26	0.54	3.90	3.23	2.23	0.45	16.31	0.047	0.52
		14:40	0.74	0.93	1.16	0.53	3.75	3.24	2.29	0.42	16.42	0.045	0.53
		15:00	0.71	0.86	1.20	0.51	3.69	3.18	2.30	0.38	15.81	0.045	0.53
		15:20	0.73	0.90	1.27	0.51	3.74	3.14	2.26	0.39	15.40	0.047	0.52
		15:40	0.76	0.98	1.33	0.54	3.51	3.18	2.26	0.41	15.81	0.048	0.52
		16:00	0.80	1.00	1.22	0.58	3.76	3.31	2.21	0.50	17.12	0.047	0.52

Location	Date	Time	H_s (m)	H_{10} (m)	H_{max} (m)	H_{rms} (m)	T_s (s)	T_z (s)	T_c (s)	$\boldsymbol{\varepsilon}$	L_o (m)	H/L	ξ_o
Site 8	15/01/11	16:20	0.90	1.08	1.48	0.65	3.86	3.48	2.23	0.56	18.87	0.048	0.52
		16:40	0.81	1.02	1.32	0.57	3.88	3.28	2.33	0.41	16.84	0.048	0.52
		17:00	0.83	1.05	1.37	0.59	3.75	3.23	2.20	0.47	16.31	0.051	0.50
		17:20	0.82	1.05	1.38	0.57	3.75	3.25	2.23	0.45	16.48	0.050	0.51
		17:40	0.83	1.02	1.33	0.59	4.15	3.51	2.34	0.50	19.20	0.043	0.55
		18:00	0.78	0.97	1.37	0.56	3.91	3.36	2.44	0.38	17.67	0.044	0.54
		18:20	0.83	1.05	1.44	0.59	3.95	3.49	2.45	0.42	18.97	0.044	0.54
		18:40	0.88	1.06	1.49	0.63	4.09	3.49	2.33	0.50	18.97	0.046	0.53
		19:00	0.90	1.13	1.42	0.63	3.98	3.44	2.33	0.47	18.44	0.049	0.51
		19:20	0.95	1.19	1.49	0.68	4.00	3.39	2.21	0.53	17.93	0.053	0.49
		19:40	1.07	1.28	1.63	0.78	4.33	3.80	2.12	0.79	22.49	0.048	0.52
		20:00	1.00	1.28	1.84	0.72	4.46	3.74	2.22	0.68	21.78	0.046	0.53
Site 5	06/02/11	12:00	0.32	0.39	0.54	0.13	3.28	2.77	2.05	0.35	12.02	0.027	0.92
		12:20	0.32	0.40	0.49	0.23	3.13	2.63	2.01	0.31	10.76	0.030	0.87
θ = 8.52°		12:40	0.34	0.41	0.52	0.24	3.24	2.64	2.09	0.26	10.85	0.031	0.85
		13:00	0.34	0.41	0.60	0.24	3.08	2.57	2.02	0.27	10.35	0.033	0.83
		13:20	0.38	0.47	0.66	0.27	2.97	2.46	1.98	0.24	9.47	0.040	0.75
		13:40	0.39	0.48	0.70	0.28	3.01	2.51	2.00	0.25	9.87	0.039	0.76
		14:00	0.30	0.39	0.55	0.22	2.90	2.43	1.92	0.26	9.20	0.033	0.83
		14:20	0.27	0.33	0.47	0.19	2.63	2.32	1.89	0.23	8.43	0.032	0.84
		14:40	0.25	0.31	0.41	0.18	2.62	2.30	1.90	0.21	8.25	0.030	0.86
		15:00	0.22	0.27	0.43	0.16	2.52	2.22	1.85	0.20	7.68	0.029	0.89
		15:20	0.22	0.28	0.38	0.16	2.40	2.19	1.84	0.19	7.48	0.029	0.87
		15:40	0.25	0.32	0.44	0.18	2.52	2.15	1.82	0.19	7.23	0.035	0.80
		16:00	0.31	0.38	0.47	0.22	2.86	2.51	1.98	0.27	9.83	0.031	0.85

APPENDIX 3
Summary Wind Data

East Weather Station
Monthly averages of Hourly Wind Data Over the Period July 2010 – February 2011

Beaufort Number	Speed ms^{-1}	July (hours)	Aug (hours)	Sept (hours)	Oct (hours)	Nov (hours)	Dec (hours)	Jan (hours)	Feb (hours)	Total (%)
0	< 0.5	3	108	49	71	53	101	74	5	10.01
1	0.5 - 2.1	66	476	199	359	413	276	325	55	46.80
2	2.1 - 3.6	2	65	89	109	108	99	105	12	12.71
3	3.6 - 5.7	2	45	108	77	47	66	70	11	9.19
4	5.7 - 8.8	3	12	128	79	52	110	113	33	11.43
5	8.8 - 11.1	2	15	80	42	18	51	32	7	5.33
6	11.1 - 14.4	5	19	37	19	7	35	22	4	3.19
7	14.4 - 17.5	-	3	14	3	4	5	3	7	0.84
8	17.5 - 21.1	-	1	9	3	-	1	-	2	0.35
9	21.1 - 24.7	-	-	4	-	-	-	-	-	0.09
10	24.7 - 28.8	-	-	3	-	-	-	-	-	0.06
11	28.8 - 32.9	-	-	-	-	-	-	-	-	0.00

South Weather Station
Monthly averages of Hourly Wind Data Over the Period July 2010 – October 2010

Beaufort Number	Speed ms^{-1}	July (hours)	Aug (hours)	Sept (hours)	Oct (hours)	Total (%)
0	< 0.5	1	39	22	14	3.37
1	0.5 - 2.1	89	536	308	391	58.79
2	2.1 - 3.6	2	87	109	146	15.28
3	3.6 - 5.7	2	45	164	79	12.88
4	5.7 - 8.8	2	14	96	45	6.97
5	8.8 - 11.1	5	5	15	7	1.42
6	11.1 - 14.4	2	15	6	3	1.15
7	14.4 - 17.5	-	3	-	-	0.13
8	17.5 - 21.1	-	-	-	-	0
9	21.1 - 24.7	-	-	-	-	0
10	24.7 - 28.8	-	-	-	-	0
11	28.8 - 32.9	-	-	-	-	0

APPENDIX 4
CliFlo Weather Station Comparison

Specific periods are highlighted and are depicted in either a sudden change in wind speed or direction.

Northerly Wind Event: 12/08/10 – 13/08/10

Date/Time	Mt Cook EWS		East WS		Tekapo EWS		South WS		Pukaki Aerodrome EWS	
	ms^{-1}	degrees	ms^{-1}	degrees	ms^{-1}	degrees	ms^{-1}	degrees	ms^{-1}	degrees
12/8 9:00	2.00	283	0.96	37	4.31	348	1.21	277	1.64	285
12/8 10:00	2.61	304	2.01	352	4.39	10	1.14	126	1.50	265
12/8 11:00	3.00	324	2.13	185	1.89	38	1.86	147	1.19	144
12/8 12:00	6.61	342	2.47	186	9.19	34	2.07	155	1.69	124
12/8 13:00	5.81	330	8.41	352	8.69	30	2.17	182	0.89	201
12/8 14:00	6.11	341	9.55	345	7.00	10	3.43	29	1.11	198
12/8 15:00	7.61	340	9.93	338	6.50	2	6.00	10	1.97	89
12/8 16:00	9.11	344	13.45	332	6.11	348	4.17	30	1.58	130
12/8 17:00	7.81	339	12.93	340	9.81	360	6.52	21	2.33	286
12/8 18:00	9.31	339	13.61	342	11.39	357	7.51	16	5.97	32
12/8 19:00	8.11	337	12.96	347	12.31	360	12.22	359	6.89	54
12/8 20:00	9.19	337	13.88	350	14.81	360	15.21	356	3.53	355
12/8 21:00	7.69	326	16.88	35	13.19	352	12.49	357	10.92	339
12/8 22:00	5.89	320	14.47	37	12.50	359	12.74	7	8.25	280
12/8 23:00	4.39	299	13.54	31	16.81	10	14.29	9	10.53	337
13/8 0:00	2.39	303	16.36	32	15.61	360	12.00	21	10.97	313
13/8 1:00	1.89	270	20.97	34	20.89	354	11.32	36	10.22	360
13/8 2:00	1.69	282	13.26	27	18.61	11	7.70	59	11.50	28
13/8 3:00	1.39	298	3.09	21	12.69	28	5.23	25	10.67	43
13/8 4:00	3.61	201	0.65	47	9.19	4	3.74	24	5.19	55
13/8 5:00	3.69	187	0.49	42	5.69	3	2.54	9	2.44	73

Light southerlies before northerly event at East WS and South WS.

Moderate northeasterlies at South WS before shifting north.

Strong northerly event.

Northeast shift following northerly event.

Southerly Wind Event: 7/08/10 – 8/08/10

Date/Time	Mt Cook EWS		East WS		Tekapo EWS		South WS		Pukaki Aerodrome AWS	
	ms^{-1}	degrees	ms^{-1}	degrees	ms^{-1}	degrees	ms^{-1}	degrees	ms^{-1}	degrees
7/8 14:00	0.61	157	1.36	189	1.50	77	0.74	316	1.81	116
7/8 15:00	0.81	355	0.84	204	1.19	134	0.96	226	2.28	158
7/8 16:00	1.39	321	3.00	200	3.69	108	2.44	216	2.25	199
7/8 17:00	0.61	204	6.31	201	5.81	114	4.82	243	3.06	274
7/8 18:00	3.50	118	7.95	199	5.39	131	4.70	239	6.83	259
7/8 19:00	7.00	134	7.17	200	5.89	145	5.10	239	7.58	261
7/8 20:00	2.39	251	6.27	195	4.61	216	4.36	241	7.78	258
7/8 21:00	1.50	286	5.47	193	4.81	232	4.54	245	7.50	256
7/8 22:00	1.00	327	6.28	196	5.00	235	4.24	244	7.50	258
7/8 23:00	0.61	12	5.72	198	4.00	229	3.87	246	6.89	267
8/8 0:00	0.89	331	5.19	197	2.81	228	3.59	246	4.22	266
8/8 1:00	0.61	15	4.64	194	2.39	231	3.68	244	5.81	264
8/8 2:00	1.61	132	5.20	197	2.81	226	3.12	245	5.00	262
8/8 3:00	1.89	174	5.35	193	2.11	220	2.94	241	3.33	228
8/8 4:00	3.50	133	5.07	192	2.61	228	3.18	241	4.03	236
8/8 5:00	4.69	135	4.57	192	2.39	232	2.90	244	3.75	257
8/8 6:00	2.69	144	4.92	190	2.61	231	3.03	244	4.19	252
8/8 7:00	0.81	214	4.73	190	2.81	231	3.28	247	3.44	269
8/8 8:00	0.00	0	4.42	190	2.89	230	3.09	243	4.69	272
8/8 9:00	0.11	336	4.14	192	3.00	230	2.97	245	5.11	271
8/8 10:00	0.11	40	3.90	185	3.31	232	2.97	244	4.83	271
8/8 11:00	0.00	0	3.53	180	3.19	232	2.90	241	4.72	257
8/8 12:00	0.11	325	3.19	180	3.61	230	2.94	234	4.56	256
8/8 13:00	0.11	309	3.71	187	4.11	228	2.84	240	3.89	255
8/8 14:00	1.50	331	3.34	185	4.11	238	2.75	240	3.75	261
8/8 15:00	1.50	310	3.84	194	3.19	234	2.20	215	1.81	331
8/8 16:00	2.69	153	3.71	195	2.81	231	2.29	218	1.97	169
8/8 17:00	2.69	132	4.92	201	2.39	210	2.29	219	1.56	21
8/8 18:00	2.89	147	4.51	195	4.39	206	1.51	220	1.69	104
8/8 19:00	5.39	128	3.18	184	3.39	202	1.83	232	2.50	273
8/8 20:00	5.61	130	1.27	141	1.69	202	2.13	226	3.19	241

Southerly event.

Variable wind speeds and directions at Mt Cook EWS.

Moderate southerly at Mt Cook EWS.

Fading southerly winds at East WS and South WS.

APPENDIX 5
Hindcasted Wave Statistics During High Lake Levels In January 2011

Definition of Symbols

A Profile A

B Profile B

LL The lake level combined with the wave runup

All other symbols are defined on pages xv and xvi.

Note: The average slope angle (β) of Profile A and Profile B is used for the calculation of Q.

Location	Date	U (ms-[1])	U hours	direction	H_s (m)	T (s)	$R_{2\%}$ A (m)	$R_{2\%}$ B (m)	LL A (msl)	LL B (msl)	α_b (rad)	Vs_w	Q x 10^{-2} (m^3h^{-1})	Gross x 10^{-1} (m^3)
Site 2	1/01	8.92	6	8.01	0.36	2.3	0.31	0.23	533.01	532.93	0.98	0.257	8.65	5.19
	2/01	8.19	5	18.47	0.3	2.1	0.26	0.19	532.83	532.76	0.98	0.245	5.74	2.87
SN = 290°	3/01	8.05	6	10.34	0.32	2.1	0.26	0.19	532.64	532.57	0.98	0.253	7.19	4.32
	4/01	6.08	5	8.20	0.23	1.8	0.19	0.14	532.49	532.44	0.98	0.232	3.33	1.66
	5/01	1.09	7	298.65	0.01	0.6	0.01	0.01	532.29	532.28	-	-	-	-
A θ =11.31°	6/01	7.63	5	14.59	0.28	2	0.23	0.17	532.45	532.39	0.98	0.243	5.09	2.55
B θ = 8.34°	7/01	10.45	5	349.82	0.47	2.5	0.38	0.28	532.59	532.49	0.96	0.281	17.65	8.82
	8/01	2.17	10	203.08	0.06	1.3	-	-	-	-	-	-	-	-
	9/01	2.18	7	215.61	0.06	1.3	-	-	-	-	-	-	-	-
	10/01	2.14	8	158.34	0.05	1.2	-	-	-	-	-	-	-	-
	11/01	2.41	5	217.01	0.09	1.3	-	-	-	-	-	-	-	-
	12/01	2.53	7	175.50	0.13	1.5	-	-	-	-	-1.55	0.191	0.86	0.60
	13/01	2.20	10	202.98	0.06	1.3	-	-	-	-	-	-	-	-
	14/01	3.38	4	13.60	0.1	1.3	0.09	0.07	531.97	531.95	0.98	0.180	0.41	0.16
	15/01	13.06	4	14.09	0.55	2.7	0.44	0.33	532.31	532.19	0.98	0.293	25.54	10.22
	16/01	9.83	6	1.10	0.42	2.4	0.35	0.25	532.22	532.13	0.98	0.271	13.34	8.01
	17/01	4.53	5	15.48	0.15	1.5	0.13	0.10	532.01	531.98	0.98	0.205	1.18	0.59
	18/01	1.87	4	9.80	0.05	1	0.05	0.04	532.01	531.99	-	-	-	-
	19/01	12.57	4	354.50	0.59	2.8	0.48	0.35	532.53	532.41	0.96	0.298	29.46	11.79
	20/01	2.33	5	177.20	0.09	1.3	-	-	-	-	-	-	-	-
	21/01	1.86	5	211.24	0.04	1.1	-	-	-	-	-	-	-	-
	22/01	2.40	9	177.92	0.08	1.5	-	-	-	-	-	-	-	-
	23/01	2.73	7	225.09	0.12	1.4	-	-	-	-	-1.55	0.190	0.77	0.54
	24/01	1.77	10	236.62	0.03	0.9	-	-	-	-	-	-	-	-
	25/01	4.69	11	18.16	0.15	1.5	0.13	0.10	532.19	532.16	0.98	0.205	1.18	1.29
	26/01	9.60	4	15.08	0.37	2.3	0.31	0.23	532.36	532.28	0.98	0.260	9.52	3.81
	27/01	6.57	5	10.60	0.24	1.9	0.21	0.15	532.30	532.24	0.98	0.231	3.38	1.69
	28/01	6.19	7	13.27	0.22	1.8	0.19	0.14	532.33	532.28	0.98	0.227	2.85	1.99
	29/01	5.78	4	7.20	0.21	1.8	0.18	0.14	532.39	532.34	0.98	0.222	2.42	0.97
	30/01	11.99	5	8.02	0.52	2.7	0.43	0.32	532.68	532.57	0.98	0.285	20.99	10.50
	31/01	10.14	5	349.38	0.46	2.5	0.38	0.28	532.62	532.53	0.96	0.278	16.37	8.18

Location	Date	U (ms^{-1})	U hours	direction	H_s (m)	T (s)	$R_{2\%}$ A (m)	$R_{2\%}$ B (m)	LL A (msl)	LL B (msl)	α_b (rad)	Vs_w	Q x 10^{-2} (m^3h^{-1})	Gross x 10^{-1} (m^3)
Site 4	1/01	8.92	6	8.01	0.44	2.5	0.23	0.24	532.94	532.94	1.20	0.316	18.57	11.14
	2/01	8.19	5	18.47	0.36	2.3	0.19	0.20	532.77	532.78	1.20	0.298	11.33	5.67
SN = 275°	3/01	8.05	6	10.34	0.39	2.4	0.21	0.22	532.59	532.60	1.20	0.304	13.48	8.09
	4/01	6.08	5	8.20	0.28	2.1	0.16	0.16	532.45	532.46	1.20	0.275	5.90	2.95
	5/01	1.09	7	298.65	0.01	0.6	0.01	0.01	532.28	532.28	-	-	-	-
A θ =7.18°	6/01	7.63	5	14.59	0.34	2.3	0.19	0.20	532.40	532.41	1.20	0.290	9.28	4.64
B θ =7.41°	7/01	10.45	5	349.82	0.59	2.9	0.31	0.32	532.52	532.53	1.20	0.340	35.77	17.89
	8/01	2.17	10	203.08	0.09	1.3	0.05	0.06	532.34	532.34	-	-	-	-
	9/01	2.18	7	215.61	0.09	1.2	0.05	0.05	532.35	532.35	-	-	-	-
	10/01	2.14	8	158.34	0.05	1.2	0.04	0.04	532.26	532.26	-	-	-	-
	11/01	2.41	5	217.01	0.09	1.3	0.05	0.06	532.14	532.14	-	-	-	-
	12/01	2.53	7	175.50	0.12	1.5	0.07	0.08	532.05	532.05	-1.59	0.213	0.76	0.53
	13/01	2.20	10	202.98	0.06	1.3	0.04	0.05	531.95	531.95	-	-	-	-
	14/01	3.38	4	13.60	0.12	1.4	0.07	0.07	531.95	531.95	1.22	0.221	0.84	0.34
	15/01	13.06	4	14.09	0.67	3.1	0.36	0.37	532.22	532.23	1.20	0.351	47.25	18.90
	16/01	9.83	6	1.10	0.52	2.7	0.27	0.28	532.15	532.16	1.20	0.331	27.49	16.49
	17/01	4.53	5	15.48	0.18	1.7	0.10	0.10	531.98	531.98	1.20	0.245	2.13	1.07
	18/01	1.87	4	9.80	0.05	1	0.03	0.03	531.99	531.99	-	-	-	-
	19/01	12.57	4	354.50	0.72	3.2	0.38	0.39	532.44	532.45	1.20	0.358	56.15	22.46
	20/01	2.33	5	177.20	0.09	1.3	0.05	0.06	532.19	532.20	-	-	-	-
	21/01	1.86	5	211.24	0.04	1.1	0.03	0.03	532.19	532.19	-	-	-	-
	22/01	2.40	9	177.92	0.12	1.4	0.07	0.07	532.20	532.20	-1.55	0.221	0.90	0.81
	23/01	2.73	7	225.09	0.11	1.4	0.07	0.07	532.16	532.17	-1.47	0.211	0.66	0.46
	24/01	1.77	10	236.62	0.05	0.9	0.03	0.03	532.10	532.10	-	-	-	-
	25/01	4.69	11	18.16	0.18	1.7	0.10	0.10	532.16	532.17	1.20	0.245	2.13	2.35
	26/01	9.60	4	15.08	0.45	2.6	0.25	0.25	532.30	532.31	1.20	0.314	18.21	7.29
	27/01	6.57	5	10.60	0.3	2.1	0.16	0.17	532.25	532.26	1.20	0.285	7.52	3.76
	28/01	6.19	7	13.27	0.27	2	0.15	0.15	532.29	532.29	1.20	0.277	5.87	4.11
	29/01	5.78	4	7.20	0.26	2	0.14	0.15	532.35	532.35	1.20	0.272	5.15	2.06
	30/01	11.99	5	8.02	0.64	3	0.34	0.35	532.59	532.60	1.20	0.348	43.69	21.85
	31/01	10.14	5	349.38	0.56	2.8	0.30	0.30	532.54	532.55	1.20	0.337	32.53	16.27

Location	Date	U (ms^{-1})	U hours	direction	H_s (m)	T (s)	$R_{2\%}$ A (m)	$R_{2\%}$ B (m)	LL A (msl)	LL B (msl)	$\boldsymbol{\alpha}_b$ (rad)	Vs_w	Q x 10^{-2} (m^3h^{-1})	Gross x 10^{-1} (m^3)
Site 5	1/01	8.92	6	8.01	0.44	2.5	0.23	0.24	532.94	532.94	1.20	0.316	18.57	11.14
	2/01	8.19	5	18.47	0.36	2.3	0.19	0.20	532.77	532.78	1.20	0.298	11.33	5.67
SN = 275°	3/01	8.05	6	10.34	0.39	2.4	0.21	0.22	532.59	532.60	1.20	0.304	13.48	8.09
	4/01	6.08	5	8.20	0.28	2.1	0.16	0.16	532.45	532.46	1.20	0.275	5.90	2.95
	5/01	1.09	7	298.65	0.01	0.6	0.01	0.01	532.28	532.28	-	-	-	-
A θ =7.18°	6/01	7.63	5	14.59	0.34	2.3	0.19	0.20	532.40	532.41	1.20	0.290	9.28	4.64
B θ =7.41°	7/01	10.45	5	349.82	0.59	2.9	0.31	0.32	532.52	532.53	1.20	0.340	35.77	17.89
	8/01	2.17	10	203.08	0.09	1.3	0.05	0.06	532.34	532.34	-	-	-	-
	9/01	2.18	7	215.61	0.09	1.2	0.05	0.05	532.35	532.35	-	-	-	-
	10/01	2.14	8	158.34	0.05	1.2	0.04	0.04	532.26	532.26	-	-	-	-
	11/01	2.41	5	217.01	0.09	1.3	0.05	0.06	532.14	532.14	-	-	-	-
	12/01	2.53	7	175.50	0.12	1.5	0.07	0.08	532.05	532.05	-1.59	0.213	0.76	0.53
	13/01	2.20	10	202.98	0.06	1.3	0.04	0.05	531.95	531.95	-	-	-	-
	14/01	3.38	4	13.60	0.12	1.4	0.07	0.07	531.95	531.95	1.22	0.221	0.84	0.34
	15/01	13.06	4	14.09	0.67	3.1	0.36	0.37	532.22	532.23	1.20	0.351	47.25	18.90
	16/01	9.83	6	1.10	0.52	2.7	0.27	0.28	532.15	532.16	1.20	0.331	27.49	16.49
	17/01	4.53	5	15.48	0.18	1.7	0.10	0.10	531.98	531.98	1.20	0.245	2.13	1.07
	18/01	1.87	4	9.80	0.05	1	0.03	0.03	531.99	531.99	-	-	-	-
	19/01	12.57	4	354.50	0.72	3.2	0.38	0.39	532.44	532.45	1.20	0.358	56.15	22.46
	20/01	2.33	5	177.20	0.09	1.3	0.05	0.06	532.19	532.20	-	-	-	-
	21/01	1.86	5	211.24	0.04	1.1	0.03	0.03	532.19	532.19	-	-	-	-
	22/01	2.40	9	177.92	0.12	1.4	0.07	0.07	532.20	532.20	-1.55	0.221	0.90	0.81
	23/01	2.73	7	225.09	0.11	1.4	0.07	0.07	532.16	532.17	-1.47	0.211	0.66	0.46
	24/01	1.77	10	236.62	0.05	0.9	0.03	0.03	532.10	532.10	-	-	-	-
	25/01	4.69	11	18.16	0.18	1.7	0.10	0.10	532.16	532.17	1.20	0.245	2.13	2.35
	26/01	9.60	4	15.08	0.45	2.6	0.25	0.25	532.30	532.31	1.20	0.314	18.21	7.29
	27/01	6.57	5	10.60	0.3	2.1	0.16	0.17	532.25	532.26	1.20	0.285	7.52	3.76
	28/01	6.19	7	13.27	0.27	2	0.15	0.15	532.29	532.29	1.20	0.277	5.87	4.11
	29/01	5.78	4	7.20	0.26	2	0.14	0.15	532.35	532.35	1.20	0.272	5.15	2.06
	30/01	11.99	5	8.02	0.64	3	0.34	0.35	532.59	532.60	1.20	0.348	43.69	21.85
	31/01	10.14	5	349.38	0.56	2.8	0.30	0.30	532.54	532.55	1.20	0.337	32.53	16.27

Location	Date	U (ms-[1])	U hours	direction	H_s (m)	T (s)	$R_{2\%}$ A (m)	$R_{2\%}$ B (m)	LL A (msl)	LL B (msl)	$\boldsymbol{\alpha}_b$ (rad)	Vs_w	Q x 10^{-2} (m^3h^{-1})	Gross x 10^{-1} (m^3)
Site 7	1/01	8.92	6	8.01	0.58	3	0.40	0.41	533.10	533.11	0.23	0.297	6.69	4.01
	2/01	8.19	5	18.47	0.47	2.7	0.32	0.33	532.90	532.91	0.23	0.282	4.17	2.09
SN = 330°	3/01	8.05	6	10.34	0.51	2.8	0.35	0.36	532.73	532.74	0.23	0.289	5.07	3.04
	4/01	6.08	5	8.20	0.36	2.4	0.25	0.26	532.55	532.55	0.23	0.262	2.20	1.10
	5/01	1.09	7	298.65	0.01	0.6	0.01	0.01	532.28	532.28				
A θ = 8.9°	6/01	7.63	5	14.59	0.45	2.6	0.30	0.31	532.52	532.53	0.23	0.281	3.94	1.97
B θ = 9.2°	7/01	10.45	5	349.82	0.77	3.4	0.52	0.54	532.73	532.74	0.17	0.322	10.18	5.09
	8/01	2.17	10	203.08	-	-	-	-	-	-	-	-	-	-
	9/01	2.18	7	215.61	-	-	-	-	-	-	-	-	-	-
	10/01	2.14	8	158.34	-	-	-	-	-	-	-	-	-	-
	11/01	2.41	5	217.01	-	-	-	-	-	-	-	-	-	-
	12/01	2.53	7	175.50	-	-	-	-	-	-	-	-	-	-
	13/01	2.20	10	202.98	-	-	-	-	-	-	-	-	-	-
	14/01	3.38	4	13.60	0.13	1.5	0.09	0.10	531.98	531.98	0.37	0.199	0.32	0.13
	15/01	13.06	4	14.09	0.88	3.6	0.59	0.61	532.45	532.47	0.23	0.334	18.25	7.30
	16/01	9.83	6	1.10	0.69	3.2	0.46	0.48	532.34	532.35	0.23	0.314	10.45	6.27
	17/01	4.53	5	15.48	0.22	2	0.16	0.17	532.04	532.05	0.26	0.224	0.71	0.36
	18/01	1.87	4	9.80	0.05	1	0.04	0.04	532.00	532.00			0.00	0.00
	19/01	12.57	4	354.50	0.95	3.7	0.63	0.65	532.69	532.71	0.17	0.343	17.19	6.88
	20/01	2.33	5	177.20	-	-	-	-	-	-	-	-	-	-
	21/01	1.86	5	211.24	-	-	-	-	-	-	-	-	-	-
	22/01	2.40	9	177.92	-	-	-	-	-	-	-	-	-	-
	23/01	2.73	7	225.09	-	-	-	-	-	-	-	-	-	-
	24/01	1.77	10	236.62	-	-	-	-	-	-	-	-	-	-
	25/01	4.69	11	18.16	0.24	2	0.17	0.18	532.23	532.24	0.23	0.234	0.84	0.92
	26/01	9.60	4	15.08	0.59	3	0.40	0.42	532.46	532.47	0.23	0.300	7.10	2.84
	27/01	6.57	5	10.60	0.39	2.5	0.27	0.28	532.36	532.37	0.23	0.267	2.63	1.32
	28/01	6.19	7	13.27	0.35	2.4	0.25	0.26	532.39	532.40	0.23	0.258	1.99	1.40
	29/01	5.78	4	7.20	0.31	2.3	0.22	0.23	532.43	532.44	0.28	0.248	1.78	0.71
	30/01	11.99	5	8.02	0.83	3.5	0.56	0.58	532.81	532.82	0.23	0.329	15.95	7.98
	31/01	10.14	5	349.38	0.74	3.3	0.50	0.51	532.74	532.76	0.17	0.320	9.55	4.77

Location	Date	U (ms-^{1})	U hours	direction	H_s (m)	T (s)	$R_{2\%}$ A (m)	$R_{2\%}$ B (m)	LL A (msl)	LL B (msl)	α_b (rad)	Vs_w	Q x 10^{-2} (m^3h^{-1})	Gross x 10^{-1} (m^3)
Site 8	1/01	8.92	6	8.01	0.69	3.2	0.37	0.43	533.07	533.14	-0.16	0.342	7.91	4.75
	2/01	8.19	5	18.47	0.59	3	0.32	0.38	532.89	532.95	-0.14	0.326	4.78	2.39
SN = 4°	3/01	8.05	6	10.34	0.61	3.1	0.33	0.39	532.71	532.77	-0.16	0.327	5.56	3.34
	4/01	6.08	5	8.20	0.14	2.6	0.13	0.16	532.43	532.45	-0.14	0.171	0.04	0.02
	5/01	1.09	7	298.65	0.01	1.6	0.02	0.03	532.29	532.30	-	-	-	-
A θ =7°	6/01	7.63	5	14.59	0.55	2.9	0.30	0.35	532.51	532.57	-0.14	0.321	4.07	2.04
B θ = 8.31°	7/01	10.45	5	349.82	0.87	3.6	0.46	0.55	532.67	532.75	-0.17	0.362	14.73	7.36
	8/01	2.17	10	203.08	-	-	-	-	-	-	-	-	-	-
	9/01	2.18	7	215.61	-	-	-	-	-	-	-	-	-	-
	10/01	2.14	8	158.34	-	-	-	-	-	-	-	-	-	-
	11/01	2.41	5	217.01	-	-	-	-	-	-	-	-	-	-
	12/01	2.53	7	175.50	-	-	-	-	-	-	-	-	-	-
	13/01	2.20	10	202.98	-	-	-	-	-	-	-	-	-	-
	14/01	3.38	4	13.60	0.14	1.6	0.08	0.10	531.97	531.98	0.07	0.218	0.08	0.03
	15/01	13.06	4	14.09	1.08	3.9	0.56	0.66	532.42	532.52	-0.14	0.387	20.59	8.24
	16/01	9.83	6	1.10	0.8	3.5	0.43	0.51	532.30	532.38	-0.17	0.352	11.78	7.07
	17/01	4.53	5	15.48	0.25	2.1	0.14	0.17	532.02	532.05	-0.03	0.254	0.14	0.07
	18/01	1.87	4	9.80	0.05	1	0.03	0.04	531.99	531.99				
	19/01	12.57	4	354.50	1.09	4	0.57	0.68	532.63	532.74	-0.17	0.384	24.91	9.96
	20/01	2.33	5	177.20	-	-	-	-	-	-	-	-	-	-
	21/01	1.86	5	211.24	-	-	-	-	-	-	-	-	-	-
	22/01	2.40	9	177.92	-	-	-	-	-	-	-	-	-	-
	23/01	2.73	7	225.09	-	-	-	-	-	-	-	-	-	-
	24/01	1.77	10	236.62	-	-	-	-	-	-	-	-	-	-
	25/01	4.69	11	18.16	0.3	2.2	0.17	0.20	532.23	532.26	-0.14	0.272	0.97	1.07
	26/01	9.60	4	15.08	0.73	3.3	0.39	0.46	532.44	532.51	-0.14	0.346	7.94	3.18
	27/01	6.57	5	10.60	0.47	2.7	0.25	0.30	532.34	532.39	-0.16	0.307	3.16	1.58
	28/01	6.19	7	13.27	0.43	2.6	0.23	0.28	532.38	532.42	-0.14	0.299	2.26	1.58
	29/01	5.78	4	7.20	0.33	2.4	0.19	0.22	532.39	532.43	-	-	-	-
	30/01	11.99	5	8.02	1	3.8	0.52	0.62	532.77	532.87	-0.16	0.378	18.87	9.43
	31/01	10.14	5	349.38	0.83	3.5	0.44	0.52	532.69	532.77	-0.17	0.359	13.40	6.70

Location	Date	U (ms^{-1})	U hours	direction	H_s (m)	T (s)	$R_{2\%}$ A (m)	$R_{2\%}$ B (m)	LL A (msl)	LL B (msl)	α_b (rad)	Vs_w	Q x 10^{-2} (m^3h^{-1})	Gross x 10^{-1} (m^3)
Site 11	1/01	8.92	6	8.01	0.56	2.9	0.58	0.43	533.28	533.14	-1.45	0.259	24.72	14.83
	2/01	8.19	5	18.47	0.49	2.7	0.50	0.38	533.08	532.95	-1.41	0.251	18.43	9.22
SN = 88°	3/01	8.05	6	10.34	0.49	2.7	0.50	0.38	532.88	532.76	-1.45	0.251	18.52	11.11
	4/01	6.08	5	8.20	0.35	2.3	0.36	0.27	532.66	532.57	-1.45	0.230	8.52	4.26
	5/01	1.09	7	298.65	0	0								
A θ = 13.55°	6/01	7.63	5	14.59	0.45	2.6	0.46	0.35	532.68	532.56	-1.41	0.245	15.04	7.52
B θ = 10.2°	7/01	10.45	5	349.82	0.65	3.1	0.66	0.50	532.87	532.71	-1.45	0.270	35.26	17.63
	8/01	2.17	10	203.08	0.04	1.1	-	-	-	-	-	-	-	-
	9/01	2.18	7	215.61	0.03	0.9	-	-	-	-	-	-	-	-
	10/01	2.14	8	158.34	0.09	1.3	0.10	0.08	532.33	532.30	1.22	0.155	0.29	0.23
	11/01	2.41	5	217.01	0.04	1								
	12/01	2.53	7	175.50	0.11	1.4	0.12	0.09	532.10	532.07	1.36	0.165	0.51	0.35
	13/01	2.20	10	202.98	0.04	1.1	-	-	-	-	-	-	-	-
	14/01	3.38	4	13.60	0.14	1.6	0.16	0.12	532.04	532.00	-1.29	0.174	0.83	0.33
	15/01	13.06	4	14.09	0.88	3.5	0.87	0.66	532.73	532.52	-1.41	0.295	74.79	29.92
	16/01	9.83	6	1.10	0.63	3	0.63	0.48	532.51	532.35	-1.45	0.270	34.30	20.58
	17/01	4.53	5	15.48	0.24	2	0.26	0.20	532.14	532.08	-1.41	0.204	3.21	1.60
	18/01	1.87	4	9.80	0.05	1	0.06	0.04	532.02	532.00	-1.36	0.132	0.07	0.03
	19/01	12.57	4	354.50	0.84	3.4	0.83	0.62	532.88	532.68	-1.45	0.293	68.67	27.47
	20/01	2.33	5	177.20	0.09	1.3	0.10	0.08	532.24	532.22	1.38	0.155	0.30	0.15
	21/01	1.86	5	211.24	0.03	0.9	-	-	-	-	-	-	-	-
	22/01	2.40	9	177.92	0.1	1.3	0.11	0.08	532.24	532.21	1.36	0.163	0.44	0.39
	23/01	2.73	7	225.09	0.06	1	-	-	-	-	-	-	-	-
	24/01	1.77	10	236.62	0.01	0.5	-	-	-	-	-	-	-	-
	25/01	4.69	11	18.16	0.25	2	0.27	0.20	532.33	532.26	-1.41	0.208	3.70	4.07
	26/01	9.60	4	15.08	0.6	3	0.62	0.47	532.67	532.52	-1.41	0.263	28.78	11.51
	27/01	6.57	5	10.60	0.38	2.4	0.39	0.30	532.48	532.38	-1.43	0.234	10.19	5.10
	28/01	6.19	7	13.27	0.35	2.3	0.36	0.27	532.51	532.42	-1.41	0.230	8.48	5.93
	29/01	5.78	4	7.20	0.33	2.3	0.35	0.26	532.56	532.47	-1.45	0.223	6.93	2.77
	30/01	11.99	5	8.02	0.8	3.4	0.81	0.61	533.06	532.86	-1.45	0.286	57.89	28.94
	31/01	10.14	5	349.38	0.63	3	0.63	0.48	532.88	532.73	-1.45	0.270	34.30	17.15